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REPORTS, PAPERS, DISCUSSIONS, AND MEMOIRS

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AUTOMOBILE HAZARD IN CITIES AND ITS REDUCTION

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Synopsis

This paper directs attention to the great variations in the hazard of automobile operation in the larger cities of the United States; shows what this variation is; derives a theoretical formula to account for it; shows the applicability of this formula to cities of the United States; and discusses conclusions resulting from the investigation, which show the great extent to which street hazard is controllable by proper city planning and zoning measures.

Introduction

In taking first place among the causes of accidental deaths in this country the automobile has created a serious accident problem. Its solution has been attempted almost wholly by the use of police or regulatory measures, rather than by the application of engineering principles. State highway engineers, it is true, study highway safety and, sometimes, give it an important place in their design. City engineers have given some attention to safe road surfaces, to the improvement of especially dangerous corners, and other such details; but, from the broader standpoint of city planning, the problem of traffic accidents has received little attention.

Yet street safety is certainly one of the problems of the city planner. It is just as important that streets be safe as that they be convenient. This is realized in an abstract way, and the principle is applied in such obvious matters as grade separation at extra hazardous intersections; but street safety has held so little place in the minds of city planners that no fundamental analysis has been made of the automobile hazard with the object of determining the extent to which it is controlled by the physical characteristics of a city, and by what characteristics.

Failure to attempt such analysis has no doubt resulted from a general impression that personal factors, such as effectiveness of police control of traffic, carefulness of pedestrians, and average driving skill, together with variations in the ratio of automobiles to population, account for such differ-

Note.—This paper is issued before the date set for presentation and discussion. Correspondence is invited and may be sent by mail to the Secretary. Discussion on the paper will be closed in **August, 1927**, and, when finally closed, the paper, with discussion in full, will be published in *Transactions*.

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ences as may exist in the hazard of automobile operation in different cities. This erroneous idea, in turn, no doubt rests on a general lack of knowledge of the tremendous variation in that hazard between cities of the same size, a variation so great and of such nature as to be accounted for only on the basis of controlling physical differences in the cities themselves rather than by personal differences in their inhabitants.

The hazard variation being what it is, discovery of its underlying cause is important because of the light it may throw on means of hazard reduction. The primary purpose of this paper, therefore, is to find this cause.

Accident Frequencies in Various Cities

The average driver of a private passenger automobile in New York, N. Y., will probably injure (or possibly kill) some one with his automobile about once in six years. If an operator goes eight years without such an accident either he drives less than the average, or he is an unusually good driver, or is exceptionally lucky. In Indianapolis, Ind., however, on the basis of present conditions, the driver of a private passenger car who is responsible for more than one personal injury accident in his entire lifetime of driving, is below par in driving skill, or drives more than the average, or is attended with worse luck.

This is not a random assertion, but a statement based on accurate records of the casualty insurance companies. For every 100 automobile public liability insurance policies (covering personal injury accidents) written in 1922 on private passenger cars by a large group of insurance carriers in New York City, 16.6 accidents involving personal injury claims resulted. For every 100 such policies written in Indianapolis, Ind., in 1922, only 2.7 such accidents resulted. Most other large cities fall between these two extremes. In Buffalo, N. Y., there were 9.6 accidents; in Chicago, Ill., 6.6; and in Detroit, Mich., 4.0, per 100 policies.*

It will be shown subsequently that the rate charged for this insurance in any large city is so established that it may be accepted as a fair criterion of the personal injury hazard of operation of a private passenger automobile. Consequently, the variation in rates from city to city measures the variation in driving hazard. Table 1 shows this variation in rates (and in hazard) for a few representative cities.

Throughout this paper the expressions, "automobile hazard" and "hazard of automobile operation", unless otherwise specifically noted, mean the hazard to which an individual automobile operator is subject, of inflicting injury on some one with his automobile. It does not mean the hazard of injury to which the operator himself or a pedestrian is subject. It is the hazard involved in driving an automobile.

Importance of Differences in Hazard

Few people realize that these great differences in driving hazard exist. The automobile accident situation has become so serious in practically every

* 1922 policy year experience, National Bureau of Casualty and Surety Underwriters, New York City.

city as to obscure this fact. These differences, however, are of tremendous importance; the annual cost of public liability insurance for all the automobiles in New York City would be in excess of $75 000 000. This may be taken as less rather than more than the economic loss from personal injuries from automobile accidents, as the administrative cost of handling this business, which of course is included in the rates charged, is offset by the facts that, in a collision between two automobiles, this insurance covers the injuries sustained by only one of the parties to the accident, and that in a collision between an automobile and a pedestrian (if contributory negligence on the part of the pedestrian is shown) the pedestrian is likely to receive little or no indemnification for his injuries. Therefore, accepting the cost of insurance as the economic loss (from personal injuries only) the annual cost of these in New York City is at least $75 000 000. If the hazard in New York were the same as that in Chicago, the resulting injury cost would be reduced to about $28 000 000 per year, and the resulting saving would be about $47 000 000.

TABLE 1.—AUTOMOBILE PUBLIC LIABILITY INSURANCE RATES, IN PERCENTAGES OF THE RATE FOR NEW YORK CITY.*

Place.	Percentage.	Place.	Percentage.
New York, N. Y.	100	Detroit, Mich.	27
Buffalo, N. Y.	52	Atlanta, Ga.	24
Philadelphia, Pa.	50	Washington, D. C.	22
Cleveland, Ohio	46	New Orleans, La.	22
Providence, R. I.	40	Los Angeles, Calif.	22
Chicago, Ill.	37	Indianapolis, Ind.	20

* Based on 1924 Manual of Automobile Insurance, National Bureau of Casualty and Surety Underwriters, New York City.

Investigation of this hazard variation with a view of determining why it exists and whether it must necessarily exist is, therefore, worth while. A superficial examination of rates shows no necessary reason for its existence. It can be noted that rates are generally higher in large than in small cities; higher in the East and North than in the West and South; and higher in old than in new cities.

There are so many exceptions, however, to each of these rules as to make it apparent that none of them controls the hazard. If the hazard is greater in large, old, Eastern cities, it is not because of their size, age, or geographical location, but rather because of some other factor which usually, but far from invariably, accompanies size, age, and certain geographical locations. The problem, then, is to discover what this unknown controlling factor is.

MEASURE OF HAZARD VARIATION

Before attempting the solution, however, the validity of the standard selected for measuring hazard variation will be explained briefly. In discussions of the automobile problem the automobile death rate in terms of population is usually invoked to show the seriousness of the situation; and, properly, because this seriousness bears a relationship to the proportion of the

population killed, rather than to the number of automobiles which did the killing. In discussing why the death rate is what it is, however, the ratio of automobiles to population cannot be neglected. Los Angeles, Calif., for example, with one car to each three persons, would be expected to have more automobile fatalities in proportion to population than New York, with one car to each fifteen persons. What is not necessarily expected is that New York, instead of having one-fifth or one-sixth as many fatalities in proportion to population as Los Angeles, should have one-half as many; in other words, that the death rate per automobile should be three times as great in New York as in Los Angeles.

This is a significant fact in the problem. Automobile use will continue and doubtless will increase. Improvement will not come about by reduction of the number of automobiles in use, but by reduction of the damage attributable to each automobile. Anything that decreases the hazard created by each automobile will decrease the total automobile hazard. Hence, the individual automobile and the hazard created by the individual automobile are the starting points, and hazard variation must be correlated with registration of automobiles rather than with population.

On this basis, the first measure of hazard which comes to mind is the ratio of automobile fatalities to automobile registrations; but the proportion of accidents that result fatally is small, and is not uniform between cities. Data from some localities indicate as many as four or five fatalities per hundred personal injury automobile accidents, whereas other localities apparently have only two or three fatalities from a hundred such accidents.* Also, for all but the very largest cities, automobile fatalities are not so numerous as to furnish fatality rates comparatively unaffected by chance, and in consequence wide variations occur from year to year. Further still, reliable figures for automobile registrations in cities are largely lacking. For these reasons, fatality rates are not available as a satisfactory measure of the hazard of automobile operation.

Insurance Rates as Measure

Automobile liability insurance rates, however, do furnish a reasonably accurate index, and practically the only index, as the larger insurance companies are the only agencies which deal with automobile accidents, both fatal and non-fatal, on a nation-wide basis. This index is not wholly satisfactory, first because not all automobiles carry insurance, and secondly, because no one insurance policy covers the entire hazard. It is, however, the best index available. There is no reason to believe that the hazard created by an insured car is materially different from that created by an uninsured car; or at least that the difference in hazard created by the two classes of cars varies materially between cities. Although separate policies are written to cover the personal injuries and the property damage resulting from automobile accidents,

* Report of Committee on Statistics, First National Conference on Street and Highway Safety, p. 11. The great deviations between the ratios given for different cities (from slightly more than one fatality per hundred injuries to six fatalities per hundred injuries) are no doubt, as noted in the report, largely due to differences as between cities in definitions of an "injury" and in methods of reporting accidents, but must also reflect considerable differences in the true ratios.

if automobile public liability rates are accepted as the index, they will broaden the scope to include the hazard of all personal injuries of any moment, whether fatal or not. Such injuries constitute the bulk of the automobile problem. These rates, as applied to private passenger cars, were used in Table 1.

Rate-Making

"Public liability" insurance is an automobile owner's insurance against legal liability for personal injuries which may result from operation of his insured car. From a somewhat haphazard system in the early days of the automobile, rate-making has reached a high degree of accuracy, so that now the public liability insurance rate of a large city may be considered a satisfactory measures of the legal liability hazard which automobile operation in that city entails. Subsequently it will be shown that the total chance of inflicting personal injury (whether the injury results in legal liability or not) is proportional to the legal liability hazard. Therefore, the variation in public liability insurance rates (Table 1) also measures the variation in the chance of injuring some one while driving an automobile.

Rates for this insurance are promulgated annually by a bureau maintained for this purpose by about thirty of the leading stock casualty insurance companies. Each member company furnishes this bureau with its loss experience, and in this way a sufficient body of experience to serve as a reliable guide is accumulated. Practically all automobile liability insurance written in the United States is based on rates established by this bureau.

For establishment of these rates, the country is divided into 251 territories (consisting each of a large city and its closest suburbs) or territorial groups (consisting each of several small cities or several counties). For each of these territories or groups separate loss experience is secured. In some cases the volume of exposure of a territory or group is not sufficiently large to be fairly indicative, and further combination is necessitated. However, for each community that develops a sufficient exposure (and this includes most of the larger cities, say, of 200 000 population and more) rates are calculated from the community's individual data and are in consequence a reflection of that community's traffic hazard.

The method of establishing the rates on private cars is as follows:* The number of cars insured in the community during each of the three preceding years is ascertained, and also the losses incurred in each of these years as indemnities for accidents attributable to the insured cars. Dividing the losses for each year by the number of cars insured, and averaging the three resulting quotients, gives the average loss per car or "pure premium". This average "pure premium" is weighted by a fixed percentage to cover the administrative cost of the business and the average rate is thus determined for each community or group. For purposes of business administration these groups are then combined into a much smaller number of "rate territories", in each of which the rate is the average of the rates of the groups included in that "rate territory". The units comprising any given territory may be widely

* For a full discussion of this subject see "Automobile Rate-Making," by H. P. Stellwagen, *Proceedings*, Casualty Actuarial Soc., Vol. XI, Pt. 2.

scattered over the country, but the average rate assessed against all localities which are placed in any one rate territory is within a very few per cent. of the rate determined individually for each group placed in that territory. Sixteen (in 1924 and 1925) average public liability rates for private passenger cars are thus established, some one of which will apply to any locality in the United States. Finally, by another set of calculations differentials are obtained which are applied to the average rate to give the actual listed rate for different makes of cars.

Thus, the variation in liability rates for private passenger cars, considering only cities large enough to be individually rated, is an accurate reflection of the public liability hazard for insured cars. There is no reason to believe that the hazard of operation of uninsured cars is materially different from that of insured cars; the variation of rates from city to city for commercial cars and public vehicles is nearly the same as for private passenger cars; and private passenger cars very largely predominate in every city. For these reasons the variation of rates for private passenger cars (Table 1) may be taken to indicate the variation in public liability hazard of all automobile operation in such cities.

What, then, is the reason for this great variation, and what is the determinant of a hazard that varies so widely from one city to another?

Theoretical Derivation of Formula for Hazard Variation

For purposes of analysis, imagine a city in which traffic, both vehicular and pedestrian, is distributed uniformly over the entire street mileage, all paved; in which no automobile accidents occur except collisions of automobiles with other automobiles and with pedestrians; in which all automobiles are of the same type, and are the only vehicles in use; and in which the percentage of automobiles in use at any given time is the same as the percentage of the population using the streets as pedestrians at that same time.

Let H = the hazard that results from the annual use in this city of the average automobile (average in respect to total annual mileage, maintenance, and driving skill).

h = the hazard that results from operation of this automobile through a unit distance.

h_a and h_p = the components of this hazard that relate to collisions with other automobiles and with pedestrians, respectively.

Then,

$$h = h_a + h_p$$

Let M = the street mileage of the city.

P = the population of the city.

R = the number of automobiles registered in the city.

a = the average number of occupants of each automobile in use.

p and r = the number of pedestrians and of automobiles, respectively, that the average automobile passes from any direction in traveling a unit distance.

As pedestrians and automobiles are uniformly distributed over the streets of the city, $p \propto \dfrac{P - a\,R}{M}$ and $r \propto \dfrac{R}{M}$.

As every time an automobile passes a pedestrian or another automobile, there is danger of a collision, obviously, $h_p \propto p$ and $h_a \propto r$.

Then,

$$h_p \propto \frac{P - a\,R}{M},\ \text{or}\ h_p = k_1 \frac{P - a\,R}{M}$$

and,

$$h_a \propto \frac{R}{M},\ \text{or}\ h_a = k_2 \frac{R}{M}$$

k_1 and k_2 being constants.

And,

$$h = k_1 \frac{P - a\,R}{M} + k_2 \frac{R}{M}$$

Let,

$$\frac{k_2}{k_1} = w$$

then,

$$h = k_1 \frac{P - a\,R}{M} + k_1 \frac{w\,R}{M}$$

or,

$$h \propto \frac{P - a\,R + w\,R}{M}$$

or,

$$h \propto \frac{P + (w - a)\,R}{M}$$

If $w = a$, (and it will be shown that from a public liability standpoint for cities of the United States this is approximately true), the expression becomes,

$$h \propto \frac{P}{M}$$

Also,

$$H \propto h\,;$$

then,

$$H \propto \frac{P}{M}$$

giving the basic relation of automobile hazard to physical surroundings.

This variation is rationally deduced and should express accurately (assuming that $a = w$) the elements of the public liability hazard of automobile operation in the hypothetical city. It does not necessarily follow that it accurately expresses the elements determining the hazard of operation in cities of the United States, because assumptions were made in the derivation of the formula which do not hold true, and the effect of these untrue assumptions on the applicability of the formula is problematical.

Test of Validity of the Formula

There is, however, the means of testing what this effect is, to what extent it invalidates applicability of the formula to American cities. The actual variation in hazard from one city to another is known from insurance rates. It has been shown that this variation covers a very wide range and that it is independent of the size, age, or geographical location of cities. The variation is also not caused by automobile congestion or by differences in the effectiveness of traffic control.

These possible causes of the variation can be quickly eliminated. If automobile congestion determined hazard, New York, with a hazard four and one-half times that of Los Angeles, would have to have a vehicular congestion four and one-half times as great, not only in the center of the city, but in residential districts as well. This is not true. In fact, some cities in which automobile use is most extensive are those in which the hazard is least.

As to the variation being caused by differences in efficiency of traffic control, any such idea is disproved by study of the variation itself (see Fig. 3). The cities having the most advanced traffic regulation, as, for example, those in States embraced in the Eastern Conference of Motor Vehicle Administrators, often show the highest, not the lowest, hazards. If excellence of traffic regulation determined hazard, the variation would be in many respects just the reverse of what it is now. This is true because good traffic regulation is usually the outgrowth of high hazard—of conditions so intolerable that they force improvement.

On the other hand, if this hazard variation can be accounted for on the basis of the formula, $H \propto \frac{P}{M}$, very strong evidence will be furnished that this formula does correctly set forth the determinants of public liability hazard. If comparison of the variation calculated from the formula with the actual variation as known from insurance rates, reveals discrepancies, these discrepancies may serve to define the limitations of the formula or to indicate modifications that must be made before it will be applicable to cities of the United States.

Modification for Unpaved Streets

One such modification of the formula is necessary before this comparison of the calculated and actual variations can be attempted. In deriving the formula it was assumed that all streets of the hypothetical city were paved. In actual cities this is not true—the ratio of unpaved to paved mileage varies between wide limits. In applying the formula it is not fair wholly to disregard this unpaved mileage, nor should it be included at its face value. In a typical city in which the unpaved mileage makes up 25 or 30% of the total, the unpaved streets are largely on the outskirts of the city, where perhaps the majority of lots are vacant and the streets do not carry the traffic that would warrant their being paved. Hence, in including them with paved mileage, they should be weighted at much less than unity to indicate their much lower mile-for-mile value in distributing traffic and population.

Considering a city, however, in which there are several miles of unpaved streets for each mile of paved streets, a large portion of unpaved streets will be in outlying, partly developed areas, but a part will also be in built-up districts and will carry almost as much local traffic as they would carry if paved. These latter streets should be included with paved streets at not much less than their face value, or if both these classes of unpaved streets be lumped together (as must be done, practically) a higher coefficient should be used to express their average weighted value than was used for the typical city in which paved mileage exceeds the unpaved. This principle can be expressed as follows:

In determining the value of M in the expression, $H \propto \frac{P}{M}$, paved street mileage should be counted at its full face value, whereas unpaved street mileage should be multiplied by a coefficient less than one, this coefficient increasing, at a slowly decreasing rate, with the increase in the ratio of unpaved streets to total street mileage.

The values to be assigned this coefficient are wholly a matter of judgment. It seems impossible to express exactly the relative importance of the two classes of streets in all cities by a single graph, but as an approximation sufficiently close for present purposes, the writer suggests Fig. 1.

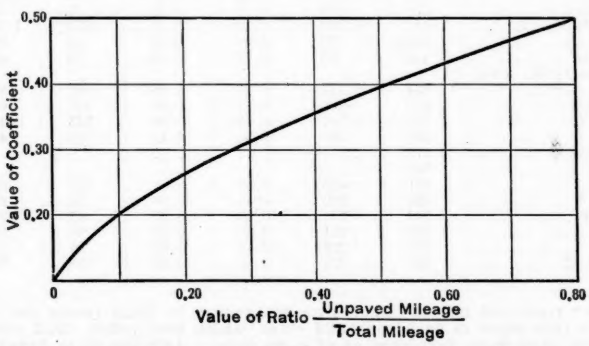

FIG. 1.—COEFFICIENTS TO BE USED IN WEIGHTING MILEAGES OF UNPAVED STREETS.

HANDLING OF WATER-BOUND MACADAM STREETS

In applying the principle of weighting unpaved mileage at less than a normal value, gravel and water-bound macadam streets present a problem. In most cities they could be classified as unpaved streets in that the surfacing is temporary, pending sufficient development to warrant improved paving. In a few instances, however, the mileage of water-bound macadam streets is so great as to lead to a belief that it is continued in use on streets which have reached their full (usually residential) development. In this paper, therefore, the term "paved streets" is restricted to include only types of paving better than water-bound macadam, except where the mileage of water-bound macadam exceeds that of the better types. In such cities (Providence, R. I.,

Denver, Colo., Worcester, Mass., and Hartford, Conn., for example) the excess of water-bound macadam over higher types of paving is considered as "paved".

Table 2 shows mileages weighted in accordance with these principles for all cities in the United States of 200 000 population and more (except Newark and Jersey City, N. J., which will be discussed later) for which the writer has been able to secure the necessary data. These mileages will be used for the quantity, M, in the formula, $H \propto \frac{P}{M}$.

TABLE 2.—STREET MILEAGES OF CITIES OF 200 000 POPULATION AND MORE.

City. (1)	Paved mileage.† (2)	Unpaved mileage.† (3)	Ratio, unpaved / Total (4)	Unpaved mileage coefficient. (5)	Weighted unpaved mileage. (6)	Weighted total mileage. (7)	Adjusted weighted mileage,* M. (8)
Akron, Ohio	143	257	0.64	0.44	113	256	256
Baltimore, Md.	657	173	0.21	0.27	47	704	694
¶Boston, Mass.	762	294	0.28	0.31	91	858	853
Buffalo, N. Y.	456	189	0.29	0.32	60	516	516
Chicago, Ill.	2 756	600	0.18	0.25	150	2 906	2 834
Cincinnati, Ohio	306	646	0.68	0.45	291‡	597‡	537‡
Cleveland, Ohio	657	298	0.31	0.33	97‡	754‡	678‡
Columbus, Ohio	340	110	0.24	0.29	32	372	359
Denver, Colo	241	659	0.73	0.47	309	550	536
Detroit, Mich.	826	600	0.42	0.37	222	1 048	1 048
Indianapolis, Ind.	558	135	0.19	0.26	35	593	593
Los Angeles, Calif.	557	1 599	0.74	0.47	751‡	1 308‡	1 163‡
Louisville, Ky.	245§	99	0.29	0.32	32	277	273
Milwaukee, Wis.	446	58	0.12	0.22	13	459	459
‖Minneapolis and St. Paul, Minn.	334	1 116	0.77	0.48	536	870	870
New Orleans, La.	203	890	0.81	0.50	445	648	618
**New York, N. Y.	1 829	1 505	0.45	0.38	571	2 400	2 400
Omaha, Nebr.	290	380	0.56	0.42	159	449	414
Philadelphia, Pa.	1 177	.....	....	....	...	.....	1 413
Pittsburgh, Pa.	518	.....	....	....	...	.....	622
Portland, Ore.	441	418	0.49	0.39	163	604	604
††Providence, R. I.	224	152	0.40	0.36	55	279	279
Rochester, N. Y.	309	191	0.38	0.35	67	376	353
Seattle, Wash.	510	271	0.35	0.33	89	599	599
Toledo, Ohio	255	239	0.48	0.38	91	346	346
Washington, D. C.	257	270	0.51	0.40	108	365	357

* The writer requested mileage figures as of January 1, 1922 (since the hazard variation discussed in this paper is based on 1924 rates, which best reflect 1922 conditions), but, in some instances, they were, furnished as of later dates. Adjustment to January 1, 1922 is as follows: Make deductions from the weighted mileage at the rate of 6% of the paved or unpaved mileage per annum, whichever figure is the smaller.

† Mileages as of the following dates: May 7, 1924, New Orleans; January 1, 1924, Chicago, Rochester, Columbus, and Omaha; January 1, 1923, Baltimore and Denver; September 1, 1922, Louisville; July 1, 1922, Los Angeles and Washington; January 1, 1922, in all other cases.

‡ Includes a 10% reduction to allow for alleys.

§ Includes 56 miles of macadam streets; estimate by A. A. Krieger, M. Am. Soc. C. E., indicated that they were entitled to such classification.

‖ Includes the Cities of Brookline, Cambridge, Chelsea, Everett, Malden, and Somerville which are included in Boston insurance territory.

¶ Minneapolis and St. Paul form one insurance territory.

** Excludes Staten Island, which is not included in New York City insurance territory.

†† Includes the City of Pawtucket, R. I., which is included in Providence insurance territory.

The data in Columns (1) and (2) of Table 2 (except for Philadelphia and Pittsburgh) were secured, if possible, from engineering departments; failing this, from municipal reference libraries or other official sources. Paved mile-

ages shown for Philadelphia and Pittsburgh are based on *Circular No. 9,* of the Asphalt Association, New York City, and are stated to come from official sources; figures include only paved mileage as defined in this paper, and are as of January 1, 1923; to convert these figures to weighted mileage as of January 1, 1922, they are arbitrarily increased by one-fifth of their value.

Values of a and of w

It remains, however, to justify the assumption made in deriving the formula—that the quantities, a and w, are approximately equal in cities of the United States. The symbol, a, represents the average number of occupants of each automobile in use. A traffic census taken by the writer in San Diego, Calif., in 1925, showed the average number of occupants of all types of cars, Sundays and week days, to be 2.06. A similar census taken at the same time in Washington, showed an average of 2.0. Apparently, it can be assumed with little error that in any city the average number of occupants is not far from 2, and this value will be used for a.

The symbol, w, represents the ratio, $\frac{k_2}{k_1}$, these constants being used in the equation,

$$h = k_1 \frac{P - a\,R}{M} + k_2 \frac{R}{M}$$

As,

$$h_a = k_2 \frac{R}{M}$$

and,

$$h_p = k_1 \frac{P - a\,R}{M}$$

$$\frac{h_a}{h_p} = \frac{k_2\,R}{k_1\,P'}$$

in which, $P' = P - a\,R$.

Now the relative importance, $\frac{h_a}{h_p}$, of the hazard of collisions between two automobiles as compared with the hazard of collisions of an automobile with a pedestrian depends jointly on the relative severity and the relative frequency of these two types of collision. Investigation of about 1 500 accidents shows practically the same public liability resulting from a collision between two automobiles as from a collision between an automobile and a pedestrian. This number of accidents was not sufficiently large to serve as an entirely reliable guide, but in the absence of more complete data equal severity may be assumed for the two types of collisions. Therefore, their relative importance depends on their relative frequency, or,

$$\frac{h_a}{h_p} = \frac{f_a}{f_p}$$

in which, f_a and f_p represent the two frequencies. Equating the two values of $\frac{h_a}{h_p}$,

$$\frac{k_2}{k_1}\frac{R}{P'} = \frac{f_a}{f_p}$$

or,

$$\frac{k_2}{k_1} = \frac{f_a}{f_p} \times \frac{P'}{R} = w$$

The next step is to consider the relative frequency, $\frac{f_a}{f_p}$, of collisions between two automobiles and collisions of an automobile with a pedestrian. For a city in which all travel was by automobile, obviously all accidents would be collisions between two automobiles (assuming only the two types of collisions here dealt with) and the ratio, $\frac{f_a}{f_p}$, would be infinity. In a city with very few automobiles in proportion to the population, practically all accidents would be collisions with pedestrians, and the value of the ratio would approach zero. Hence, if the abscissas of a set of co-ordinates represent values of $\frac{P'}{R}$ and the ordinates represent values of the ratio, $\frac{f_a}{f_p}$, this ratio will be expressed by a graph tangent to the Y-axis at plus infinity, and to the X-axis at plus infinity. Reliable data for the determination of intermediate points are lacking, as such data involve not only complete reports of both fatal and non-fatal personal injury accidents over a considerable period to establish the ratio, $\frac{f_a}{f_p}$, but also accurate figures for automobile registrations to establish the ratio, $\frac{P'}{R}$*. It is difficult to obtain trustworthy figures covering all these quantities in any one city. The writer has secured what appear to be fairly reliable figures for $\frac{f_a}{f_p}$, however, for ratios of $\frac{P'}{R}$, which are probably about 3.0, 6.2, and 14.0. These points are plotted on Fig. 2. On Fig. 2 is drawn also a hyperbolic curve such that any ordinate multiplied by its corresponding abscissa (that is, $\frac{f_a}{f_p} \times \frac{P'}{R}$, the product being the quantity, w), gives a value of 2. This curve fits the three plotted points (both co-ordinates to which, it will be recalled, are of somewhat doubtful accuracy) with sufficient exactness to permit the acceptance of 2 as the approximate value of w. This is especially true as the quantity, R, in the expression,

$$H \propto \frac{P + (w - a) R}{M}$$

* These registrations, moreover, must be accurately divided into private passenger, commercial, and public passenger vehicles, since these latter two classes carry higher average rates than the first. Since P is much greater than R and since the ratio of "equivalent private passenger cars" to "actual private passenger cars" is about the same for most cities, this refinement can be neglected in the main formula without introducing errors greater than 1 or 2 per cent. It cannot, however, be omitted in the determination of w without introducing prohibitive errors.

is so much smaller than P for most cities that a considerable error in the value of the coefficient of R has a comparatively small effect on the value of H. Within the limits of exactness atfempted in this paper, then, it may be assumed that $w = 2$, and, hence, that $a = w$. The expression,

$$H \propto \frac{P + (w - a) R}{M}$$

thus reduces to $H \propto \frac{P}{M}$.

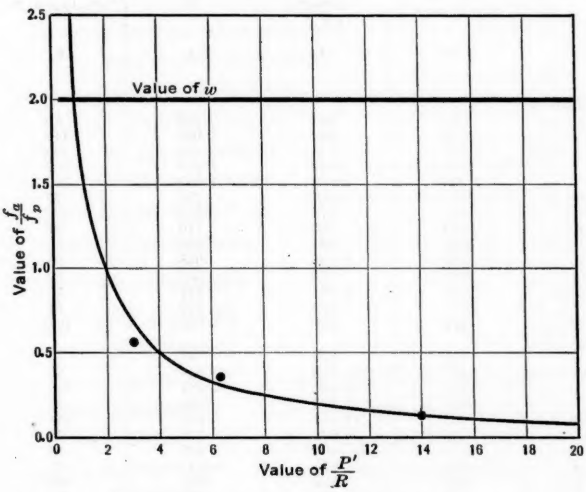

FIG. 2.—VALUES OF $\frac{f_a}{f_p}$ (RELATIVE FREQUENCIES OF ACCIDENTS TO AUTOMOBILES AND PEDESTRIANS) AND OF $\frac{f_a}{f_p} \times \frac{P'}{R}$ (OR w) FOR VARIOUS RATIOS OF $\frac{P'}{R}$ (PEDESTRIANS TO AUTOMOBILES).

"ACTUAL" AND "CALCULATED" HAZARD VARIATIONS

Proceeding with the comparison of the variation in public liability hazard as calculated from the formula with the actual variation shown by insurance rates, Table 3 shows these variations for all the cities included in Table 2. The Chicago rate is taken as the base, as percentages of which the other rates are shown. The population, P, (Column (2)) is from the the Census Bureau estimates of 1922.

The variation shown by insurance rates (Column (6) in Table 3) will hereafter be referred to as the "actual variation" in hazard, and the rate of any city (expressed as a percentage of the Chicago rate) will be termed its actual hazard. The hazard variation as determined from $H \propto \frac{P}{M}$ (Column (5) in Table 3) will hereafter be referred to as the "calculated variation" and the hazard it shows for any city (expressed as a percentage of the Chicago hazard) will be termed the "calculated hazard" of that city.

Fig. 3 is a graphic comparison of the actual and calculated variations shown in Columns (5) and (6) of Table 3. To show the diversity in size of

the cities entering into the comparison, the two variations are projected against a background proportional in width to the populations of the respective cities.

TABLE 3.—CALCULATED AND ACTUAL VARIATIONS IN HAZARD.

City. (1)	Population, (*P*), in thousands. (2)	Weighted street mileage, (*M*). (3)	$\frac{P}{M}$ (4)	Calculated variation.* (5)	Actual variation.* (6)
†Akron and Toledo, Ohio	469	602	779	78	78
Baltimore, Md.	762	694	1 098	110	80
‡Boston, Mass.	1 150	853	1 350	135	125
Buffalo, N. Y.	528	516	1 024	102	142
Chicago, Ill.	2 833	2 834	1 000	100	100
Cincinnati, Ohio	405	537	753	75	75
Cleveland, Ohio	855	678	1 262	126	125
Columbus, Ohio	255	359	710	71	64**
Denver, Colo.	268	536	500	50	54
Detroit, Mich.	994	1 048	948	95	75
Indianapolis, Ind.	335	593	566	57	54**
Los Angeles, Calif.	635	1 163	547	55	59
Louisville, Ky.	257	273	942	94	80**
Milwaukee, Wis.	477	459	1 039	104	64
§Minneapolis and St. Paul, Minn.	641	870	737	74	80
New Orleans, La.	400	618	647	65	59
‖New York, N. Y.	5 715	2 400	2 381	238	273
Omaha, Nebr.	201	414	486	49	54**
Philadelphia, Pa.	1 895	1 413	1 341	134	137
Pittsburgh, Pa.	608	622	978	98	100
Portland, Ore.	269	604	446	45	59
¶Providence, R. I.	309	279	1 108	111	109
Rochester, N. Y.	312	353	884	88	92
Seattle, Wash.	316	599	528	53	59**
Washington, D. C.	438	357	1 227	123	59

* As percentages of the value for Chicago.

† Loss experience for these two cities is combined to get an indicative exposure for determination of their insurance rate (the calculated hazard for Akron alone would be 81 and for Toledo alone would be 75).

‡ Includes the Cities of Brookline, Cambridge, Chelsea, Everett, Malden, and Somerville.

§ Minneapolis and St. Paul form one insurance territory.

‖ Exclusive of Staten Island, which is not included in New York City insurance territory.

¶ Includes the City of Pawtucket.

** Determined from local loss experience combined with similar experience in smaller cities near-by, owing to insufficient volume of local loss experience.

AGREEMENT OF VARIATIONS—CITIES OF GROUP 1

The cities included in Fig. 3 have been divided into two groups. In Group 1, consisting of twenty-one cities in nineteen rate-making units, the average discrepancy between actual and calculated hazards is 6 per cent. Considering the seventeen cities (fifteen units) of the group for which the insurance rate is based on individual experience (and for which it is certain that the insurance rate is the true, rather than the approximate, actual hazard), the mean deviation is only 5 per cent. No one of these latter cities has a deviation of more than 10%, except New York City which shows a deviation of 13%, in the direction which would be anticipated from topographical and other peculiarities of that city and that insurance territory.

Any inaccuracies in the mileage figures used will hardly increase the mean deviation by more than 1 or 2%, if at all. As the expression, $H \propto \frac{P}{M}$, accounts for the hazard variation in three-fourths of the cities investigated, its applicability to the average city of the United States under 1922 conditions seems evident.

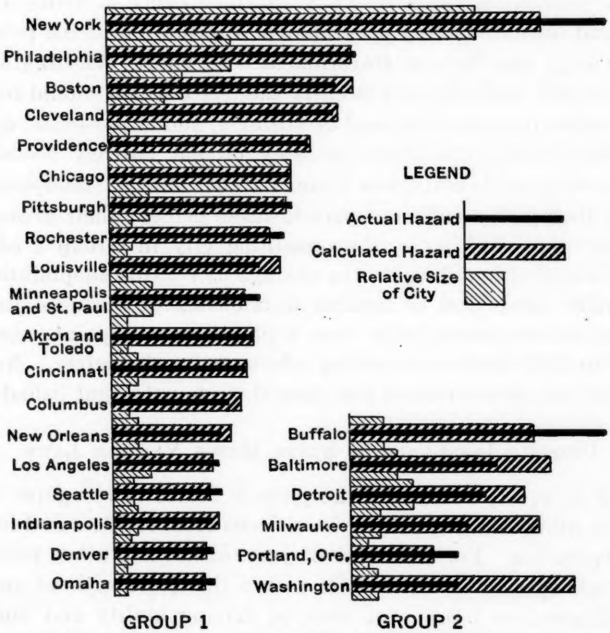

FIG. 3.—ACTUAL AND CALCULATED HAZARD VARIATIONS (FROM TABLE 3).

DISCREPANCIES IN VARIATIONS—CITIES OF GROUP 2

Referring to Group 2 of six cities, these show discrepancies between actual and calculated hazards ranging from 24% of the actual hazard for Portland, Ore., to 109% for Washington. A full discussion of the reasons underlying these discrepancies is beyond the scope of this paper. The formula, $H \propto \frac{P}{M}$, is applicable to actual cities only because the large number of factors which it fails to take into consideration seem usually to compensate for one another. Where the formula fails to apply, these factors fail to compensate, perhaps because of some one outstanding deviation from the average or of a number of minor variations all chancing to fall in the same direction. The deviations in the cases of Washington and Baltimore, however, suggest important conclusions.

Washington, D. C.—Washington shows the greatest discrepancy between actual and calculated hazards of any city of Fig. 3, the actual hazard being

less than one-half the calculated. One explanation that immediately presents itself is the superior planning of the city. This includes the street layout, with its liberal widths of pavement, its set-back building lines, with consequent good sight distances at intersections, and its straight, continuous thoroughfares, almost free from "bottle-necks" and well distributed to carry the traffic of all sections of the city. In addition, it includes the satisfactory distribution of population which results from the absence of slums and crowded tenements and the satisfactory geographical distribution of the principal business of the city, the Federal Government. In short, Washington possesses a system of streets well designed to carry the traffic which should result from a good distribution of population and of business, and it possesses such a distribution. While these possessions probably do not entirely account for the favorable position of Washington from a traffic hazard standpoint, it seems certain that they play a sufficient part to make evident their great value.

Baltimore, Md.—Baltimore, the remaining city in Group 2 of Fig. 3, is an old city, which although up to the average as regards adaptability of street layout to traffic needs and as regards distribution of business and of population, is not an exceptional city from a planning standpoint; nor had it as a city prior to 1923 made outstanding efforts for traffic safety. Nevertheless, it is subject to an actual hazard less than three-fourths that calculated for it.

Possible Influence of State Motor Vehicle Laws

Maryland, however, was one of the pioneer States to recognize the responsibility of the State in safeguarding its citizens from reckless and incompetent automobile operation. For a number of years it has had a law providing that no citizen shall operate an automobile within the State without an operator's license conditioned on passage of tests in driving ability and knowledge of traffic regulations; and also that the State Commissioner of Motor Vehicles shall be empowered to exercise a general supervision over automobile accident conditions. Within recent years a number of States have enacted such laws. Such enactments, and vigorous administration under them, however, do not date back far enough to have affected 1922 conditions, except in Maryland, Connecticut, Massachusetts, and New Jersey. Hazard conditions in these States may serve to show whether there is support for the suggestion which Baltimore conditions make that such laws are effective in reducing the hazard of automobile operation below what it otherwise would be; and the extent to which the expression, $H \propto \frac{P}{M}$, which seems to hold good generally as between cities without this special type of legislation, applies between cities in States with this type of legislation.

Few cities in these States have populations of 200 000 or more for comparison with those already studied. Connecticut has no cities of this size, but has three cities with populations of 100 000, or more—Bridgeport, Hartford, and New Haven. These cities have the same insurance rate, derived from their combined loss experience (compare Akron and Toledo in Table 3). Since

their actual hazard is the same, the three cities may be combined in this paper for comparison of both their calculated and actual hazards.

Similarly, eight cities in Massachusetts with populations of less than 200 000 are combined into one or more groups for determination of their insurance rate. The writer is not advised whether this hazard is determined from one grouping or several, but as all the cities have the same rate, it will be proper to consider them in one group for comparison of their actual and calculated hazards. These cities are Fall River, Haverhill, Lawrence, Lowell, Lynn, New Bedford, Springfield, and Worcester. Boston, of course, will be considered separately, as its actual hazard is different from that of the grouped cities.

Baltimore is the only city in Maryland of sufficient size to warrant investigation in this connection.

New Jersey has two cities—Newark and Jersey City—with populations of more than 200 000. These cities were not included in Tables 2 and 3, nor in Fig. 3 because they are not their own metropolitan centers, but form part of the Metropolitan Area of New York City (U. S. Census Bureau classification) and this has a disturbing influence on their street hazard, which is thus somewhat indeterminate. This effect results principally from the fact that, owing to the suburban location of these cities, the ratio of daytime population to resident population differs from that of the average city. In the average city there is an influx from surrounding suburban and rural areas every morning, which flows out again at night, so that the daytime (and hazard-producing) population is greater than the resident population. It is assumed that this influx is proportional to the population of a city, in which case it has no effect on the expression $H \propto \frac{P}{M}$. This assumption apparently holds true for cities which are their own metropolitan centers, but in suburban insurance territories like Newark and Jersey City, the flow of population is outward in the morning and back again at night, so that in them there is a smaller, instead of a larger, daytime than resident population. This causes less congestion on the streets and lower hazard than there would be if the cities were isolated.

This effect is heightened in such cities, and particularly in Newark, by the fact that all the suburban area about New York City is divided into sections, each of which constitutes one insurance territory, although it may contain several cities. Thus, Jersey City territory contains Bayonne and several smaller cities and towns of Hudson County, and Newark territory contains Montclair, Summit, the Oranges, and other commuting centers.

A contrary effect to that just described results from through traffic, bound in and out of New York City and traversing these suburban areas. This adds to street congestion in these areas, and partly compensates for the outflow of commuters. The compensation is more complete in Jersey City territory than in Newark territory, as the former is less of a commuting area, and, also, being closer to New York, is traversed by more through traffic.

The net effect of these suburban conditions is indeterminate, but that there is such an effect, and that it is greater in the Newark territory than in the Jersey City territory should be kept in mind.

Table 4 shows populations, weighted street mileages, the ratio, $\frac{P}{M}$, and calculated and actual hazard variations (based on Chicago) for the cities which have been mentioned as being in States which in 1922 had the better type of motor-vehicle legislation.

The street mileages shown are paved and unpaved mileages weighted and combined in accordance with principles set forth previously. The figures for the larger cities are from local sources, usually engineer departments; those for the smaller cities are usually from local sources, but are supplemented in some instances by data from The Asphalt Association, with estimates of the mileage of unsurfaced streets. The error in the mileage figures should in all cases be less than 10 per cent. Hoboken, in Jersey City territory, and Irvington and Bloomfield, in Newark territory, are omitted from Table 4 because of lack of mileage data. The population figures are from 1922 Census estimates.

TABLE 4.—CALCULATED AND ACTUAL HAZARD VARIATIONS FOR CITIES IN CONNECTICUT, MARYLAND, MASSACHUSETTS, AND NEW JERSEY.

Cities.	Population, P, in thousands.	Weighted street mileage, M.	$\frac{P}{M}$.	Calculated variation.*	Actual variation.*
Connecticut:					
Bridgeport					
Hartford	452	420	1 075	108	78
New Haven					
Massachusetts:					
†Boston	1 150	853	1 350	135	125
Fall River					
Haverhill					
Lawrence					
Lowell	945	818	1 154	115	78
Lynn					
New Bedford					
Springfield					
Worcester					
Maryland:					
Baltimore	762	694	1 098	110	80
New Jersey:					
‡Jersey City	463	255	1 818	182	125
§Newark	580	476	1 220	122	80

* As percentages of the value for Chicago.

† Includes Cities of Brookline, Cambridge, Chelsea, Everett, Malden, and Somerville.

‡ Includes Bayonne, West Hoboken, and West York, which are included in Jersey City insurance territory.

§ Includes Montclair, East Orange, Orange, West Orange, and Summit, which are included in Newark insurance territory.

INDICATED BENEFITS OF STATE CONTROL

Table 4 shows calculated hazards considerably greater than the actual hazards in all cases. For the Connecticut cities, for Baltimore, and for the smaller Massachusetts cities, the discrepancies between the two hazards are 28, 27, and 32%, respectively, of the calculated hazards. The discrepancy for

Boston is 7% of the calculated hazard, for Jersey City, 31%, and for Newark, 34 per cent.

As already noted the suburban location of Jersey City and Newark gives them unusual advantages in regard to the hazard of automobile operation, which have the effect of making actual hazards perhaps 5% lower in Jersey City and 10% lower in Newark than they would be if these cities were isolated. Making this allowance, the remaining discrepancies between their calculated and actual hazards become 28% of the calculated hazards in both cases. Then, for all the cities or groups in Table 4, except Boston, approximately the same discrepancy—about 28% of the respective calculated hazards—can be accounted for on the basis of a beneficent influence exercised by their motor-vehicle laws. As this discrepancy is so nearly the same in all cases, the following conclusions would be justified (except for Boston):

1.—As between themselves, the variation in public liability hazard of automobile operation is given by the expression, $H \propto \frac{P}{M}$.

2.—In comparison with other municipalities, cities in States having the better laws show this hazard to be from 25 to 30% lower in proportion to population congestion.

The fact that Boston has a calculated hazard only 7% in excess of the actual hazard still must be explained. As already noted Washington benefits tremendously from a well-planned street layout. Boston, by reason of topography, age, and random growth many decades ago, suffers from street conditions which in many of the older parts of the city are more or less the antithesis of those from which Washington benefits. It would seem logical, therefore, that Boston should have an actual hazard larger than its calculated hazard by perhaps 20 per cent. Instead it is 7% smaller. This discrepancy of 27% is attributable to the State motor-vehicle law. Hence, Boston tends to confirm rather than to upset the conclusions just stated.

The first conclusion supports what the first group of cities in Fig. 3 indicated, namely, that the great determinant of the public liability hazard resulting from automobile operation is population density. Of the two conclusions just stated, this is the more important from an engineering standpoint—perhaps from any standpoint.

The second conclusion—that a well-drawn and well-administered motor-vehicle law is capable of greatly reducing this hazard—is of great importance, to every motorist and to every citizen. It is of particular importance to city or State officials concerned with accidents on streets and highways.

It is interesting to note what this decrease in hazard means in terms of money. According to the National Automobile Chamber of Commerce, the registration of automobiles in Massachusetts in 1924 was 486 952 private passenger cars and 83 626 commercial and public automobiles, the increase in registrations from 1922 to 1924 being 48 per cent. If it is conservatively estimated that the increase in registrations from 1924 to 1926 was only 25%, the 1926 figures will be 618 000 private passenger cars and 105 000 commercial

and public vehicles. The average private passenger car public liability insurance rate in Massachusetts is about $30. The average commercial or public vehicle rate is more than twice this, or at least $65. The cost of public liability insurance in standard amounts for every automobile in the State for 1926 would thus be about $25 000 000. As has been pointed out this is less rather than more than the economic loss from automobile personal injuries, which will approximate $25 000 000 for the year. The automobile hazard level in Massachusetts cities has been shown to be only about 0.7 as high as the level which would be anticipated for them. If this difference between actual and anticipated levels prevails throughout the State, the anticipated personal injury loss for the year would be about $36 000 000, $11 000 000 more than it actually is. This $11 000 000 represents a saving from decrease in personal injuries only, and, of course, is accompanied by a saving from decrease in property damage from automobile accidents. Placing this latter saving at less than half as much, or $5 000 000, which seems conservative, the annual automobile accident loss in the State is $16 000 000 less than would be expected.

The annual cost of administration of this law is about $1 000 000 ($648 725.90 for 1922, fiscal year).

"Natural" Variation in Hazard

It may be said that $\frac{P}{M}$ is the determinant of the "natural" level of public liability hazard resulting from operation of an automobile, and that the variation as between cities given by the expression, $H \propto \frac{P}{M}$, is the "natural" variation in this hazard, that is, the variation that would obtain if all cities were on a par as regards street layout and methods of traffic regulation.

Transition from Public Liability to Total Personal Injury Basis

Up to this point discussion has been limited to public liability hazard, because of the lack of any reliable check on the applicability of a theoretical formula to actual hazard conditions except the variation from city to city of automobile public liability rates. As has been shown these rates vary closely with the public liability hazard of automobile operation, which, consequently, is accounted for by the expression, $H \propto \frac{P}{M}$. This same variation, however, holds good not only as regards public liability hazard, but also for the hazard of all personal injuries which automobile operation may entail.

The expression, $H \propto \frac{P}{M}$, was related to public liability hazard; when in determining the value of w it was stated that as regards public liability the average collision between two automobiles is of the same severity as the average collision between an automobile and a pedestrian.

Since when two automobiles collide the total personal injuries inflicted by each, the car responsible for the accident and the car not responsible, upon

occupants of the other, tend to be equal, the total personal injury from a collision between two automobiles tends to be twice the public liability. In a collision between an automobile and a pedestrian, public liability means legal liability of the motorist for injuries to the pedestrian. If the accident resulted wholly from some fault of the pedestrian, legal liability of the motorist does not exist; if it resulted from joint carelessness of both parties, the motorist's liability is at least curtailed.

It is impossible to determine the proportion of collisions between automobiles and pedestrians for which pedestrians are responsible. The smallest degree of responsibility is given by the Massachusetts Registry of Motor Vehicles, which reports pedestrian liability as from 35 to 40% of all automobile accidents involving pedestrians. Similar reports from New York State indicate about 50% responsibility; and from New York City (Police Department) and the State of Connecticut, somewhat more than 50 per cent.

Probably one-half, or a little less, of the personal injury to pedestrians by insured automobiles is indemnified. In other words, the total personal injury damage from such accidents tends, as in collisions between two automobiles, to be about twice the public liability. Assuming this, it follows that from either a public liability or a total personal injury standpoint, the two types of collision are of the same average severity, and the expression, $H \propto \frac{P}{M}$ holds good for both.

There are other automobile accidents in cities, such as collisions of automobiles with trains and electric cars, with fixed objects, etc. The total is small, however, compared with the two great classes, collisions between automobiles and of automobiles with pedestrians. Moreover, this small disturbing influence is doubtless in such direction as to be counteracted by any excess (of which there is probably a little) of liability of pedestrians for accidents involving them above what has been assumed for them.

It may be safely stated that the determinant of the total personal injury hazard which results from operation of an automobile in any city is given by the expression, $H \propto \frac{P \pm n R}{M}$, in which, n is a fraction probably varying a little as between cities, but always so small that the quantity, nR, is of negligible importance in comparison with P. In other words, from the standpoint either of public liability or of all personal injuries, the natural determinant of hazard of automobile operation is simply the density of population, it being practically immaterial to a motorist from a personal injury hazard standpoint, whether this population, or the proportion of it using the streets at any given time, is encountered as pedestrians or in automobiles.

Automobile Fatalities

As regards automobile fatality hazard, however, this is not true. Under unvarying methods and efficiency of traffic control and of public education in traffic safety, and with no change in the ratio, $\frac{P}{M}$, of a city during a period of

years, there should be no change in the personal injury hazard of automobile operation. If, however, during this period the ratio of automobiles to population is increasing, the ratio of automobile fatalities to automobile registrations should steadily decrease; that is, the fatality hazard of automobile operation normally decreases with an increase in the number of automobiles.

Death Rate per One Thousand Automobiles Registered

To demonstrate this: The expression for personal injury hazard of automobile operation, $H \propto \frac{P}{M}$, is derived from,

$$H \propto \frac{P - (a\,R - w\,R)}{M}$$

in which, $a = 2$ and $w = \frac{f_a}{f_p} \times \frac{P - a\,R}{R}$, which product is also approximately equal to 2 for all ratios of $\frac{P - a\,R}{R}$. In other words, whatever the degree of motorization of the city, the second term of the numerator cancels, leaving the population density, $\frac{P}{M}$, as the determinant of personal injury hazard.

Considering the fatality hazard, which may be termed, H_F, the expression may be derived,

$$H_F \propto \frac{P - (a\,R - w_F\,R)}{M}$$

in which,

$$w_F = \frac{F_a}{F_P} \cdot \frac{P - a\,R}{R}$$

F_a and F_p representing the numbers of motorists and pedestrians killed in automobile accidents, respectively. Then,

$$H_F \propto \frac{P - a\,R + \left(\frac{F_a}{F_p} \cdot \frac{P - a\,R}{R}\right) R}{M}$$

or,

$$H_F \propto \frac{\left(1 + \frac{F_a}{F_p}\right)(P - a\,R)}{M}$$

For the average city, a may be taken as 2, the expression thus becoming,

$$H_F \propto \frac{\left(1 + \frac{F_a}{F_p}\right)(P - 2\,R)}{M}$$

The expression for the variation of personal injury hazard could have been written in similar form, namely,

$$H \propto \frac{\left(1 + \frac{f_a}{f_p}\right)(P - 2\,R)}{M}$$

Here, however, as the importance of R relative to P increases, the increase of $\frac{f_a}{f_p}$ (Fig. 2) tends to neutralize it giving the simpler form, $H \propto \frac{P}{M}$.

However, the ratio of fatalities of the two types, that is, $\frac{F_a}{F_p}$, for a given ratio of population to automobiles has not the same value as has the collision ratio, $\frac{f_a}{f_p}$ (Fig. 2). Thus, in 1922, the ratio of collisions between automobiles to collisions with pedestrians, $\frac{f_a}{f_p}$, was apparently greater than 1.0 in Los Angeles, whereas the ratio of motorists killed to pedestrians killed, $\frac{F_a}{F_p}$, appears to have been as low as 0.33 or 0.25. Similarly, in New York City, the ratio, $\frac{f_a}{f_p}$, was about 0.12 in 1922, and the ratio, $\frac{F_a}{F_p}$, was 0.07. The ratio, $\frac{F_a}{F_p}$, always has lower values than $\frac{f_a}{f_p}$, and its values increase much less rapidly than those of $\frac{f_a}{f_p}$ from less highly to more highly motorized cities. The increase in the first factor of the numerator of the expression,

$$\frac{\left(1 + \frac{F_a}{F_p}\right)(P - 2\,R)}{M}$$

is insufficient to counterbalance the decrease in the second factor as the motorization increases; consequently, the fatality hazard decreases with increase in motorization, the population density, $\frac{P}{M}$, remaining constant.

The formula for H_F is entirely theoretical and neglects the effects of chance and other possible factors. The importance of these effects is indeterminate, as it is impossible to make a satisfactory check of the fatality hazard formula against the automobile death rate (as the personal injury hazard formula, $H \propto \frac{P}{M}$, was checked against the insurance rate). This is true because (*a*) the automobile fatality figures available through the Census Bureau are for deaths occurring in the respective cities, irrespective of the place of occurrence of accident, and a high automobile fatality rate in a city may indicate good hospital facilities (attracting persons injured outside) as much as bad traffic conditions; (*b*) there are apparently no figures for automobile registrations available for some cities, and, in other cities, the registration figures appear to be of doubtful accuracy; and (*c*) figures are almost wholly lacking for the ratios of motorists killed to pedestrians killed, in the several cities.

The great fluctuation in the automobile fatality rate from year to year in most cities indicates that chance plays a considerable part, as would be expected. This part may be even so great as to make it not permissible to speak

of a "probable" fatality hazard. As attempts have been made, however, to account for the variation in automobile fatality rates between cities by means of formulas, the writer suggests the formula for H_F as being apparently more applicable than any other that has come to his attention.

Empirical Formula

The principle that increase in motorization lowers the fatality hazard of operation of each automobile, may be clearly expressed by the empirical formula,

$$H_F \propto \frac{P}{M}\left(\frac{P}{R}\right)^{\frac{3}{8}}$$

This formula appears to give almost as satisfactory results as the more rationally deduced expression,

$$H_F \propto \frac{\left(1 + \frac{F_a}{F_p}\right)(P - 2\,R)}{M}$$

Such application of both these formulas as can be made in the light of present information reveals discrepancies for several cities due to some other unknown factor, probably the average speed of traffic. Cities with unusually slow average speeds seem to have fatality rates lower than would be anticipated from the formulas, and *vice versa.*

It may be noted that the average speed of traffic should have much less influence on personal injury hazard than on fatality hazard; low speeds lead to carelessness and many accidents of minor severity, whereas high speeds cause fewer accidents, but of higher individual severity, the total severity being much the same in both cases. This applies primarily to cities in which low speeds are caused by extreme congestion, especially of pedestrians, and in which high speeds are made possible through special traffic regulations, such as the "boulevard" stop plan.

Automobile Death Rate per 100 000 Population

The formulas for fatality hazard relate to the operation of the individual automobile; that is, to the hazard as expressed by the automobile death rate per thousand automobiles registered. To express the variation in the automobile death rate per 100 000 population, these formulas can be written:

$$H_{FP} \propto \frac{\left(1 + \frac{F_a}{F_p}\right)(P - 2\,R)}{M} \times \frac{R}{P}$$

and,

$$H_{FP} \propto \frac{P}{M}\left(\frac{P}{R}\right)^{\frac{3}{8}}\left(\frac{R}{P}\right)$$

or,

$$H_{FP} \propto \frac{P^{\frac{3}{8}}\,R^{\frac{5}{8}}}{M}$$

in which, H_{FP} represents the automobile death rate per 100 000 population. In these formulas, strictly speaking, R, except in the term, $(P - 2R)$, should denote "equivalent private passenger cars"; that is, the actual registration of private passenger cars plus the registrations of commercial and public vehicles weighted in such a manner as to reflect the excess of hazard which the latter create over that of the average private car. This is a refinement difficult to apply and one that may be neglected without invalidating the principle of the formulas.

Conclusions

From city planning and engineering standpoints three conclusions may be based on this study of street hazard.

Zoning to Prevent Congestion.—The variation expressing the personal injury hazard of automobile operation as proportional to population density, $\frac{P}{M}$, adds a warning against such degrees of congestion of population as prevail in almost all older large cities. This congestion will occur in newer cities and grow still worse in old cities unless the tendency is checked by zoning regulations.

Since the desirability of such zoning measures is evident from other considerations than that of street safety, it might be said that this warning is unnecessary. This is hardly a sound argument, however, because anything which may serve to show the danger of population congestion is worth while from the standpoint of its influence on public opinion, on which zoning measures must ultimately rest.

To change the population density of a city is a slow process, however. It attacks the hazard problem from a preventive standpoint, but offers little cure for existing bad conditions. The second conclusion relates to such a cure.

Traffic Segregation.—From the derivation of $H \propto \frac{P}{M}$, it is seen that this expression holds good as the determinant of hazard because the ratio, $\frac{P}{M}$, measures, for the average city, the frequency of crossings of the path of an automobile by other automobiles and by pedestrians. In other words, diffusion of population leads to a decrease in the number of possible collisions per unit distance of travel, and thereby reduces hazard. Any other measures which do the same should likewise reduce hazard. Such measures are to be found in a proper segregation of street traffic.

"Traffic segregation" is no new term, but is ordinarily used in a very limited sense to mean separation of slow-moving from fast-moving vehicular traffic. As regards safety, this is the least important of three forms of traffic segregation.

The second form is the segregation of right-angled streams of vehicular traffic from one another at intersections. This may be a space segregation, resulting from a separation of grades, or a time segregation. One of these forms is always present except when an accident occurs. Space segregation

being usually out of the question, the reduction of collisions between automobiles rests largely on a more certain time segregation. As the majority of collisions between automobiles in the average city occur at a comparatively small number of intersections, proper time-segregation measures may be quite effective in reducing the hazard of collision between automobiles. The situation in almost any city is susceptible of improvement through such means.

The third and most important form of traffic segregation, however, is given least attention. This form is the segregation of pedestrian from vehicular street use, and its importance in any community as compared with vehicular segregation at intersections is measured by the ratio of collisions with pedestrians to collisions between automobiles. Thus, in New York City, the prevention of collisions with pedestrians is, from a personal injury standpoint, at least five times as important as the prevention of collisions between automobiles, and as regards fatalities it is seven or eight times as important.

At present, about the only special effort made to reduce accidents involving pedestrians is through "jay-walking" campaigns and educational work. Such campaigns are meeting with considerable success, but the problem is sufficiently serious to merit supplementing them by careful studies to see whether a better segregation of pedestrian and vehicular street use and travel may not be accomplished. Although improved methods of handling traffic at street intersections may greatly reduce the hazard of collisions between vehicles, they will have little effect on collisions between automobiles and pedestrians, only a small proportion of which occur at street intersections.

A complete segregation of pedestrian and vehicular traffic is not possible, and would be undesirable because of the restrictions it would place on street use, or the great expense it would entail. In practically any city, however, a much safer distribution of pedestrian and vehicular travel might be realized at small expense and with facilitation rather than restriction of traffic.

To go into detailed methods of doing this is beyond the scope of this paper, but many applications of the principle may be discovered in any city. As most automobile collisions with pedestrians occur in residential districts, particular attention should be given to the segregation of pedestrian from automobile traffic in those districts, as far as possible through measures which will naturally drain the greatest possible proportion of automobile traffic into arterial ways.

A study of the high hazard areas shown on the automobile fatality map maintained by the Police Department of New York City illustrates very strikingly the great hazard from intermittent vehicular traffic through tenement streets filled with playing children. Under such conditions, even elaborate measures to bring about a better segregation of vehicles from pedestrians are justified. The roping off of "play streets" barred to vehicle use, in such areas, is the first step. The logical supplement to this is the provision of some means through which other near-by streets may be kept clear, between curb lines, of pedestrians (particularly of children at play) so that vehicles may pass through them at good rates of speed and without high hazards.

Extreme conditions may even justify the erection of fences along the curb lines of certain main traffic arteries through congested tenement districts. Such fenced streets would have to be provided with tunnels or bridges for cross traffic at intervals. These would be of great value from a safety standpoint and might prove of little inconvenience if traffic were heavy in two directions and light at right angles. They should always be the supplement of "play streets" or other adequate play areas.

In many cities where automobile hazard conditions are very bad, the regulation of traffic by police and motor-vehicle departments seems to be approaching the limits of its effectiveness. Further improvement must apparently come largely from engineering measures to insure a safer distribution of pedestrians and automobiles over the street system. Such measures imply better traffic segregation, particularly of vehicles from pedestrians, and secondarily of conflicting streams of vehicles at street intersections. A case in point is furnished by the railroad grade crossing, which for some reason attracts excessive attention as a highway hazard, with a consequence that money out of proportion to the resulting benefits has been devoted to this elimination.

It seems not to be generally realized that the sore spot in the traffic accident situation is the crowded residential or tenement district, that for every person killed or injured at a grade crossing, in the more urban States at least, ten are killed or injured in the slums and near slums.

Street Plan.—The third conclusion, of particular importance to the future growth of cities, results primarily from the deviation from anticipated hazard which the City of Washington presents. The conclusion is that a well-planned street layout with adequate provision of long, straight, broad and well located traffic thoroughfares, particularly if supplemented by measures that will bring about a good distribution of homes and business, will reduce street hazard far below its anticipated $\frac{P}{M}$-level.

Motor Vehicle Legislation.—These three conclusions as to zoning and other engineering means of street hazard reduction are supplemented by a fourth, equally important not only to engineers, but to all citizens—that street traffic hazard is responsive to proper efforts for increase in carefulness in street use by pedestrians and motorists. The enactment and efficient administration of the type of State motor-vehicle law outlined in this paper may, and as experience has proved does, reduce street and highway hazards 25 to 30 per cent.

WATER SUPPLY FOR ARMY RAILWAYS IN FRANCE

By Paul M. LaBach,* M. Am. Soc. C. E.

Synopsis

This history of water supply development, principally for United States Army railway transportation in France during the World War, 1917-1919, outlines briefly the factors ordinarily taken into consideration in problems of supply and distribution of boiler water for commercial railway purposes.

Then follows a discussion of the difficulties encountered in changing the water supply and distributing systems of several light traffic commercial railways into suitable installations for a unified military railway with very dense traffic.

In conclusion, an outline of the organization of forces used in the work, is given, together with a discussion of recommendations for an organization which would aid in simplifying and expediting similar operations.

Introduction

One of the fundamental requirements for railroad operation is water for boiler use. All division points where engines are turned or stored must have small power plants and facilities for washing out boilers. In addition there must also be roadside stations between terminals so located that the locomotive tenders may be filled at suitable intervals, the intervals or spacing being largely governed by the nature of the traffic as well as its density. Where through traffic only is handled the spacing may be longer than on lines carrying a large proportion of local freight which requires frequent train stops and much switching. The design of the locomotive also enters into the problem. Where superheaters are used with large tenders, as in modern practice, considerably longer intervals may be economical for through traffic than was formerly allowable.

On most of the world's steam railways the development of equipment for furnishing water to locomotives has been gradual. When railways are first built the traffic is usually light and the coal and water problem is not a very serious one in the aggregate, although neither can be neglected. As traffic increases, however, the size of both coal and water stations must also expand in proportion to the ton-miles per hour per mile of single track. Whether the interval between water stations must increase or decrease with the increase

Note.—Written discussion on this paper will be closed in **August, 1927.** When finally closed the paper, with discussion in full, will be published in *Transactions*.

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in traffic depends on local conditions and, as mentioned, the design of the power used. The intervals for freight traffic are also different from those required for passenger service. The problem at terminals is based on considerations which have little in common with road service.

In most countries heavy traffic railways have grown up gradually and there has been time to adjust coal and water facilities to the needs of the moment. These needs are seldom static and frequent changes are taking place under ordinary operating conditions. The general term, "operating conditions", covers a multitude of factors. Without some actual practical knowledge of daily train movements over a considerable period of time little can be told as to the practicability of any specific scheme of improvement. Other factors besides those stated enter into the subject of spacing, such as proximity of passenger stops, telegraph offices, signal blocks, passing sidings, grade crossing, and adverse grades.

The fuel problem, although serious, is not as exacting as that of water; coaling stations can be placed farther apart and coal can be stored for use at a future time. It is seldom practicable to store water for a longer period than one day on account of the bulk and resultant tankage required. The weight of the coal used is only about one-fifth that of the water. Coal can be stored in large quantities, hauled in ordinary cars, and put at convenient location for use without regard to its place of origin.

Coal Handling

In France during the period of American participation in the World War all coal was unloaded at the base ports by the different colliers, shipped to the points required, and unloaded in piles on platforms from which it was shoveled into the tenders. Although this is not the best economic method according to American standards it was effective. This has always been a common practice in Europe where labor is cheap.

Rainfall and Water Supply

In a well-watered country, such as the States of New York and Pennsylvania, water can be found, as a rule, within a reasonable distance of stations, which may be located to conform to operating conditions. The problem of finding a suitable supply is largely an economic one, unless the quality of the water is objectionable. In Western United States one railroad company has to go more than 100 miles for water that is usable. Therefore, in using the term, "water", the word, "pure", should be also added when it is to be used for locomotive boiler purposes. "Pure" in this sense has a special meaning, best defined by the use of negatives. Pure water will not cause pitting, corrosion, boiler scale, or foaming in switch engines.

It would be difficult to persuade a soldier of the American Expeditionary Force that water was not always easily available in large quantities in France, but such was the case. The exclamation of a returned soldier at a movie of some event over there, taken in the rain a year or two after his return, was "Good Lord, it ain't stopped raining yet!" In many minds this was and still is the popular idea of the French climate. Then there is the expression "sunny

France". A sunny country is usually an arid country. Which statement is true? Both are true in a sense, depending on the season. There is a wet season and a dry season. For agricultural purposes the country is all well watered, but that is largely due to humidity. The average annual rainfall varies from 600 mm. (24 in.) to 1 500 mm. (59 in.) within the area of railroad operations. Within the space of 100 km. (62 miles) the variation is from 700 to 1 000 mm., etc.

Each water station was dependent on an entirely separate source of supply. The quantity available was always governed by local conditions. The climate also had to be considered, that is, the ratio of wet to dry seasons. The basis of computation had to be the dry-weather yield, as that was the period of maximum effort anticipated. The geology and topography in the vicinity of any proposed station also frequently added their share of difficulties to the problem.

All roadside water stations of the kind contemplated are practically of equal importance unless they are close enough together for a train to skip one. In this case there was so little material for construction available that it was impossible to make the more extensive installations.

The coal and water needed for the proposed traffic scheme were in direct proportion to the tonnage to be hauled. This statement seems axiomatic, but the lack of appreciation of this fact was the cause of much of the failure of those not familiar with such matters to understand what would happen when a double-track line was given the maximum tonnage it could carry without slowing down train movement to take water. This meant that the station spacing had to be as far apart as practicable and the supply adequate at all times. The channel ports and the railroads north and east of Paris were reserved for the use of the British and French Armies. This left the Port of Brest and the ports on the Bay of Biscay for the American Army. The Ports of Marseilles and Cette in the Mediterranean were in use for transportation to Africa and the Orient. As the route from America was through Gibraltar where the submarine hazard was very great, and also the distance much greater, these ports were not much used except in special cases, until the western ports had become crowded.

Railways of the American Army Area

At the beginning, the so-called American Sector centered on Toul although the American Army operated subsequently in many other sections and had divisions as far west as Flanders on the British front. To bring the necessary supplies into the Toul Sector by rail was the object of the railroads herein discussed.

The First Line of Communication planned (Fig. 1) started at the base port of St. Nazaire and extended to Neufchateau and Epinal in the Toul Area. In addition, it included the line from the Ports of La Pallice and Rochefort which connected at Saumur with the first line, and also the line from Bordeaux to Bourges. It will be noted (Fig. 1) that if the lines at the base ports were congested there would be a greater density in the line east of Bourges. For this reason the Second Line of Communication, extending

from Bourges to Neufchateau by way of Clamecy, was adopted. The Third Line of Communication started at Tours and reached Neufchateau by way of Orleans. Other main lines leading eastwardly from Brest and northwardly from Marseilles will be noted.

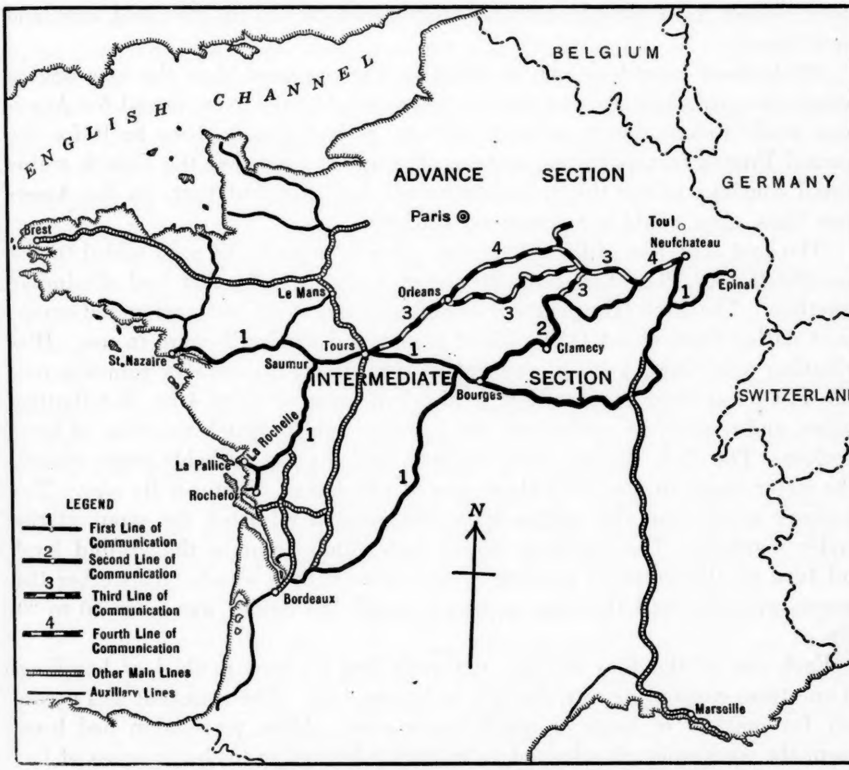

FIG. 1.—LINES OF COMMUNICATION, A. E. F., NOVEMBER, 1918.

The First, Second, and Third Lines of Communication, shown as continuous in Fig. 1, are in reality composed of parts of secondary lines (except east from Brest and north from Marseilles) of the four great systems of French railways, the Etat, Paris-Orleans, Paris-Lyons-Mediterranean, and Est. The six railway systems of France are the Nord, Est, P.-O., Etat, P. L. M., and Midi. With the exception of the Midi, which runs east and west in the south of France, all the systems radiate from Paris. The branches lie on either side of the main line so that the territory is divided into sectors with Paris as a hub. Paris, however, is not near the center of the country. The First Line of Communication with its general east and west direction is composed of secondary lines of four of the principal systems, namely, Etat, P.-O., P. L. M., and Est. The main lines of all these systems could carry very heavy traffic, although the branch lines have never been equipped for that purpose. Fortunately, they were nearly all double-tracked and the roadbeds were substan-

tially built for permanent use. Where the east and west lines crossed the main arteries to and from Paris, the terminals were usually joint. They were located for the Paris-bound train movement and were not spaced with regard to the east and west traffic. This latter feature was of considerable importance as will be shown later. A comparatively small increase in traffic on the Paris lines, which were already crowded, threatened to tie up proposed east and west lines.

With these considerations in mind it was apparent that the new engine terminals, yards, sidings, and water stations would have to be spaced for American traffic independently of such utilities as had already been built for the normal French transportation system. It was expected that the French trains would continue to use the present terminal facilities, but that, on the American lines, they would use American stations.

The fact that four different railway systems were to be used added to the complications. They had been developed independently and had dissimilar practices. The Etat (Government owned and operated) had antiquated equipment dating from about 1850. Hand pumps and windmills were in use. Distributing pipe lines, 4 in. in (approximated) diameter, were in common use. The P.-O. was more modern, but it also had some of these 4-in. distributing mains, and none of its equipment was large enough for quick watering of locomotives. The P. L. M. was quite modern, and the Est probably came second. The water crane in use everywhere was of old design, whatever its size. The engineer would stop the engine while the fireman adjusted the spout at the tender manhole. The engineer would then climb down to the ground level and turn on the water by opening a gate-valve with a wheel. Altogether the process was slow and the time necessary to fill the tender was from 10 to 30 min.

Each one of the four systems naturally had its own method of handling all questions concerning any changes in its property. Their consent was necessary for matters of large or small importance. After permission had been given, the companies also desired to be kept informed as to the progress of the work. This involved a large number of conferences with their officials. Usually a number of their men were present to pass on such points as might arise. A knowledge of colloquial French was indispensable; to this had to be added the technical equivalents for engineering terms and special apparatus to be used, little of which could be found in any published dictionary. The way to learn a proper name was usually to point out the object to a Frenchman and inquire its name and gender. The desire of the French officials to help was generally manifest.

Methods of Computation

At the beginning the only basis of calculation for water supply had to be made from a more or less hasty inspection. This showed, beginning at the west, a coastal plain, then river valleys, and, finally, territory that was mountainous. Adjacent streams would drain areas, ranging from a few, to hundreds of square miles, which might be anything from a swamp or permeable sands to steep slopes with impervious soil. The rainfall might range from

600 to 1 500 mm. As time progressed another variable was found; rain falls 150 days a year on the west coast of France, whereas the average is only 55 days at Marseilles. The change is not uniform throughout the intervening territory.

At first, there was considerable difficulty in estimating the tonnage to be hauled and the number of trains to be watered at each point. The quantity, 60 lb. per soldier per day, landing at the base ports, was used. One-third of this would be unloaded before the advance section was reached. From this beginning, the freight to be moved was calculated. The evaporation curves and horse-power curves were then used to get a tonnage rating of the American locomotives on the French tracks. The French profiles were in the metric units and the other figures in English units. A tentative tonnage rating was made and, fortunately, a trial was possible. The actual tonnage was only 3% less than the theoretical rating made by the Engineer Water Supply. The number of French trains was known, so on each line trains were added, to the point of saturation, which was assumed as being 1 train each way for each 15 min.

The accelerated growth of American participation in the war was another variable in the problem, as to quantities of water for transportation at different points. The problem would usually be presented to the Engineer Water Supply in this guise: How much water will be needed at a number of points (probably 25) when there are 500 000 soldiers in France? How much for 1 000 000? How much for 1 500 000? These questions had the advantage of being easier to ask than to answer. There were more unknown quantities than equations.

The record shows that the basis of 500 000 men in France was still being used as late as May 6, 1918. This can be explained by the fact that many of those in high command had doubts whether a large army would ever get across the Atlantic. Published diaries of some of the officers in high command show this very clearly; naturally the same view was reflected by subordinates.

The "get-together" spirit evolved by Ludendorf's drive in March, 1918, resulted in an expansion of American ideas on the subject and larger figures than hitherto used were common. By July, 1918, 4 000 000 men in France by July, 1919, was used as a basis by the Engineer Water Supply. By November, 1918, 2 000 000 men were already there.

The rate of increase in the Army was never officially announced. It had to be assumed by the person making the estimate. It was necessary to keep pace with these changing conditions until at the last the assumption was made that the Army would advance as far as the Rhine. (Fig. 2.) The rapid retreat of the German Army in October, 1918, indicated that it would be necessary to move the former broad-gauge rail-heads from the places they had occupied for several years to new locations across the devastated areas evacuated by the enemy.

Inquiries were made to determine which railroad lines would be available for the movement of supplies for the American Army in an advance to the Rhine. This program was supplemented with estimates of material and loca-

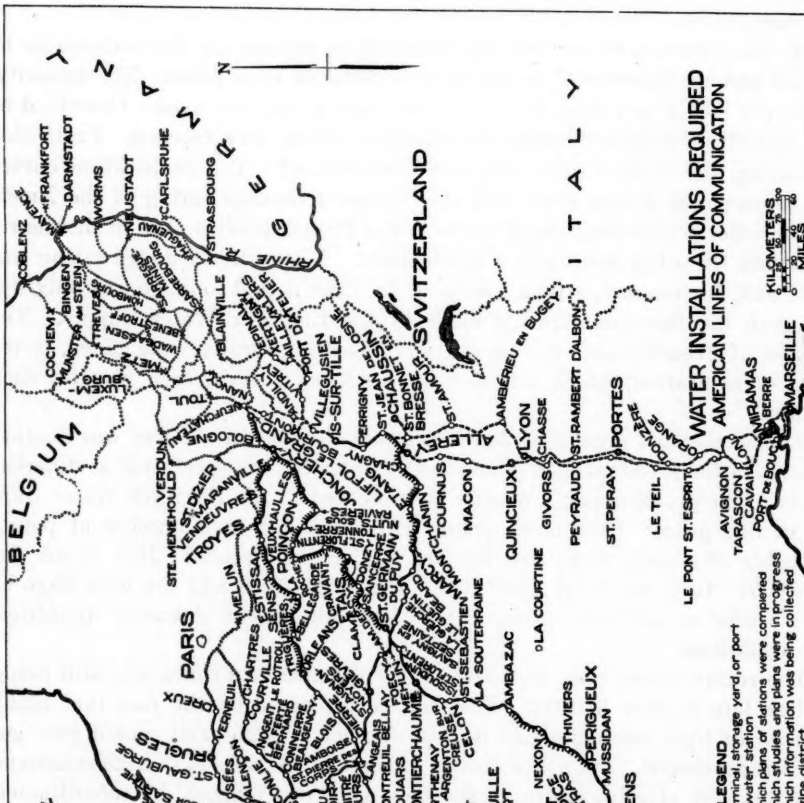
WATER INSTALLATIONS REQUIRED
AMERICAN LINES OF COMMUNICATION
LEGEND
Engine Terminal, storage yard, or port
Roadside water station
Lines on which plans of stations were completed
Lines on which studies and plans were in progress
Lines on which information was being collected
Helper engine district
KILOMETERS
MILES
BELGIUM
GERMANY
SWITZERLAND
ITALY
SPAIN
ENGLISH CHANNEL
ATLANTIC OCEAN
RHINE
PARIS
LYON
MARSEILLE
BREST
ST. NAZAIRE
LA PALLICE
LA ROCHELLE
ROCHEFORT
BORDEAUX
BASSENS
LE MANS
TOURS
SAUMUR
NEVERS
TROYES
ST. DIZIER
VERDUN
TOUL
NANCY
METZ
STRASBOURG
COBLENZ
FRANKFORT
DARMSTADT
WORMS
CARLSRUHE
GIVORS
MACON
TOURNUS
AVIGNON
MIRAMAS
ORANGE
MONTOIR
DONGES
CHARTRES
DREUX
MELUN
CHATEAUROUX
PERIGUEUX
LIMOGES
ORLEANS
BLOIS
MONTARGIS

FIG. 2.

tions for water stations which would be needed to replace those destroyed by the retreating enemy. These locations are shown on Fig. 2. Every effort was made to procure all this material well in advance of the necessity for its use.

At this time also, much information was collected and assembled, from which plans were made for water stations on the advanced lines. The studies of available sources were well advanced when the signing of the Armistice put an end to the big army program. A few installations were made in order to carry out the terms of that agreement.

Previous Preparation

Not until one year after the United States entered the war was a department organized that was responsible for all water supplies to be used in the Transportation Service. In April, 1918, the Department of Engineer of Water Service, under the Engineer of Construction, Transportation Service, was formed with the writer (then Major), and H. Malcolm Pirnie, M. Am. Soc. C. E. (then Captain), as Engineer and Assistant Engineer of Water Supply, respectively.*

The new Department was a good many laps behind the other departments which were fortunate in being able to make some preliminary strides before the rush began. However, the need for water for transportation had not been omitted in calculations for future needs.

Engineer Stores Requisition No. 6, prepared July 14, 1917, stands as the keystone in the arch of important steps absolutely needed to carry out the Transportation Service program in France. This requisition ordered material needed immediately for an army of 500 000 men. A monthly supply to be added to this without further requisition was also provided. In addition, this requisition was to be increased in proportion to any additions to the Army above 500 000 men. During the seventeen months from the time this requisition was made until the Armistice, when the proposed strength of the Army had been increased eight times, this requisition demonstrated the remarkable accuracy of its details.† This was true of large items as well as the smaller ones which included railroad water supply material.

Researches show that, apparently, this requisition is the first mention made of a railroad water supply problem. For an army of 500 000 men, an initial stock of thirty 50 000-gal. tanks and sixty water cranes was specified, with an additional supply of five tanks and ten cranes each month. For an army of 1 500 000, the monthly supply was to be increased to fifteen tanks and thirty cranes, and 4 000 000 men called for forty tanks and eighty cranes each month. If this material had arrived in France according to program, there would have been no difficulty in building proper water service installations up to and including the final advance to the Rhine.

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* The Department was afterward augmented by the addition of F. D. Nash, M. Am. Soc. C. E. (then Lieutenant). During the entire period, Sergt. J. T. Brinkley maintained a corner in a barracks called an office, with the aid (part of the time) of Private Thompson. Five was the maximum force on this work while due to illness and other causes the average was about three.

† This requisition was made under the direction of W. J. Wilgus, M. Am. Soc. C. E. (then Colonel).

Inter-Departmental Activities

From July, 1917, to April, 1918, many changes were taking place in the organization of the A. E. F., during which water supply for transportation was handled as a side issue in connection with yards, docks, and terminals. The Superintendent of Motive Power, located at Nevers, in Central France, made estimates of the quantity of water needed for mechanical purposes. The location was made by the Engineer of Construction, Transportation Service, H. C. Booz, M. Am. Soc. C. E. (then Colonel).

In January, 1918, the organization was a little more complete as regards other work, but the Water Service was still in confusion. The Transportation Department, one of the technical services of General Headquarters, was charged with the operation and maintenance of railroads, canals, wharves, road, shops, and other appurtenances needed for transportation. The small force of engineer troops in France at that time called for a division of responsibilities. The construction was under the charge of the Chief Engineer Office, Lines of Communication, another technical division of General Headquarters. At the head of the Transportation Department, was the Director General and several Deputy Directors General. Also, there were other officers with duties similar to those found on most American railways. These were General Manager, Superintendent of Motive Power, Business Manager, and Engineer of Construction. The last named was responsible for water supply with other duties in connection with the transportation program. However, under the Chief Engineer Officer, Lines of Communication, there was a Department of Water Supply which planned and constructed water-works for hospitals, camps, and towns. Under the Engineer of Construction water supply had only been considered in connection with certain definite projects which had passed the preliminary stages. These were docks, yards, and terminals.

Under the circumstances, the large development of future roadside water stations was unknown and left for later consideration. A division of the work was then made between the Director General of Transportation and the Chief Engineer Officer, Lines of Communication, in which the responsibility for the design and construction of supply works for transportation was given to the Chief Engineer Officer.

Under this ruling the Director General of Transportation wrote a letter to the Chief Engineer Officer, Lines of Communication, January 5, 1918, submitting plans of eight yards and terminal projects prepared by the Engineer of Construction. This letter included estimates by the Superintendent of Motive Power, as to water desired. In this letter is found a statement that had a great influence on future inter-department relations.

"3.—We would suggest for your approval that your Water Supply Department furnish all water requirements, piping same to the connection to the water tanks, and that we erect the tanks and provide all the piping from the tanks to the service outlets, such as water columns, ash pits, round-houses, etc. Such an arrangement will allow the one Water Supply Department to consider our requirements in connection with those of other branches of the service, and to equate the existing or new sources of supplies to the total needs."

This was immediately agreed to and left the Engineer of Construction with authority over the design of distributing systems only. As it turned out, more than 75% of the railroad water supply systems planned were at points at which there were no camps, hospitals, or other army activities; hence, the reason for the division of authority did not hold except for the projects then considered. However, this idea was expanded to include various other matters and resulted in the duplication of investigations and calculations by two departments.

ORGANIZATION OF WATER SUPPLY DEPARTMENT FOR TRANSPORTATION

The Department of Engineer of Water Supply was organized April 4, 1918. The organizers and re-organizers had gotten to work and on March 12, 1918, had given all the old authority of the Director General of Transportation to a Chief of Utilities who reported to the Commanding General, Services of Supply, who, in turn, reported to the Commander-in-Chief. The Department of Construction and Forestry was created, which took over all construction matters and the procurement of material required for construction purposes. The Engineer of Construction, Transportation Department, however, still retained the power to design, but had no control over construction. Thus, at the outset, the Engineer Water Supply* could design only the distributing systems while the Department of Construction and Forestry had the authority to investigate and design the supply systems for which it was to procure the material and construct the complete water stations.

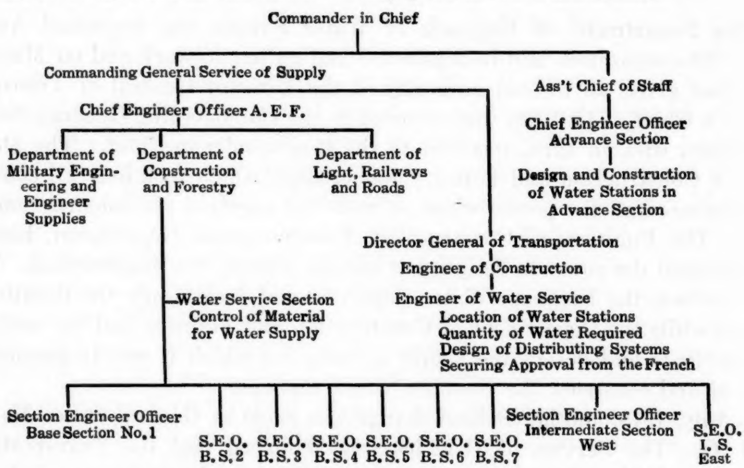
In July, 1918, another radical change was made by General Order No. 114, G. H. Q. The Service of Utilities was abolished and the Transportation Service established in much the same position it formerly occupied with respect to design, but construction was vested in the Department of Construction and Forestry. General Order No. 29, S. O. S., July 12, 1918, announced the re-organization of the Office of the Chief Engineer Officer, A. E. F., giving the Department of Construction and Forestry as one of three subdivisions. The Director of Construction and Forestry then proceeded to reorganize. The area was divided into sections each in charge of a section engineer officer who handled all construction matters, together with requisitions for materials. The Water Supply Section of the Department of Construction and Forestry only passed upon its requisitions and helped expedite the procurement of materials. The organization was de-centralized. The Transportation Department on the contrary had its authority highly centralized, as transportation was a continuous process from the sea coast to the front. (Fig. 3.)

The New Transportation Corps Water Department found itself in a peculiar position as far as its authority and responsibility were concerned. The following is quoted from an official report:

"It had authority to state when and in what quantities water would be required for railroad purposes and to design the distributing systems from the points of delivery of the supply to and including the railroad service, but it

* The term, "Engineer Water Supply", and the pronouns used as equivalents refer throughout to the entire Department as a whole and not to the head in person.

did not have authority to select the sources of supply, to specify what kind of supply equipment should be installed, or to superintend the construction in any part of the water stations. However, it was responsible for the selection of adequate sources, installation of ample supply equipment and the proper and timely construction of all water stations to insure the train operation incident to the carrying out of the transportation program for which the Director General of Transportation was responsible. In other words, the Engineer of Water Supply was responsible not only for the correct locations and proper capacities of railroad water services, but also for providing for the services an adequate supply of good water at all times, but he could not determine the source of supply, the kind and size of supply equipment, procure the material or direct the construction of any part of the installation."



Example of Procedure According to Organization.

1.—Engineer of Water Supply prepares letter from Director General of Transportation to Department of Construction and Forestry stating 200 000 gal. of water per day required at Lothiers. Letter signed by Engineer of Construction by direction of Director General of Transportation.

2.—Received by Department of Construction and Forestry.

3.—Referred to Water Supply Section for action.

4.—Indorsed over to Section Engineer Officer, Intermediate Section West.

5.—Section Engineer Officer sends engineer on ground to investigate.

6.—Engineer reports development is not practicable.

7.—Section Engineer Officer indorses report back to Department of Construction and Forestry.

8.—Referred to Water Supply Section for preparation of indorsement to Director General of Transportation.

9.—Indorsed letter received by Director General of Transportation and referred to Engineer of Construction.

10.—Engineer of Construction refers letter to Engineer of Water Supply showing development of Lothiers impracticable.

The result is that a new spacing of water stations must be devised planning to skip Lothiers; and the above procedure resorted to again.

Fig. 3.—Diagram of Organization Affecting Railroad Water Stations.

Method of Authorizing Work

Before work could be begun on either new or supplemental water stations, authority had to be had from the Commanding General, Service of Supply, G-4, U. S. A., and the French railroad on which the facility was located. A blanket approval was given by the Commanding General, Service of Supply, G-4, for all railroad water stations. The French railway companies had to be approached in either one of two methods—by official channels with consequent delays or by conferences with the French railway officials.

The official method was a roundabout course, involving complete working plans before anything could be started. The Engineer Water Supply would submit his plans to the Deputy Director General of Transportation at Paris. It then went to the Franco-American Mission for its approval, and thence to the French railway officials. With any suggested alterations, it was then returned to the point of origin through the same channels. The Engineer Water Supply through the Director General of Transportation then directed the Department of Construction and Forestry to begin construction. With this procedure, which might take a month, practically no progress could be made for obvious reasons. Knowledge of the available sources of supply and the details of the existing equipment were absolutely necessary before the spacing of the proposed water stations could be made. Detailed descriptions of the proposed equipment and method of placing it were demanded by the French railway officials before their consent could be given. Designs of distributing systems were required by the Department of Construction and Forestry in order that it might investigate the source of water and make requisitions for material for the supply works to be subsequently built by the Section Engineer.

In more ways than one this was getting "the cart before the horse". The Engineer of Water Supply was obliged to investigate sources of supply and all available materials before he could turn a wheel if he wished to provide for the train operation required. The elements involved would be:

1.—Inspection of existing water stations with their sources of supply.

2.—Inspection of proposed sites with their sources of supply. (The geology and contour of the country being an important item.)

3.—Procurement of detail plans and other information for all French water stations on the different French railways included in the Lines of Communication.

4.—Translation from the French and tabulation of the results.

5.—Studies of these and other available water sources; computation of the drainage areas to determine whether the supply would be adequate. These were to be made after inspection on the ground with the help of such records of stream flow as were available.

6.—Determination of the best spacing of water stations to meet the proposed American operations and still to conform to locations where adequate sources of supply were available within reasonable distance.

7.—Determination of the quantities required at each water station for the constantly increasing transportation problem.

8.—Submission of this information to the Department of Construction and Forestry for its investigation. (French tanks were to be used where available.)

9.—Preparation of plans for both new and reinforced water stations.*

10.—Submission of these designs to the different French railways with all available information as to the probable mechanical equipment.

* The term, "reinforced water station", was applied to French water stations which did not have adequate velocity of flow at the water crane and had to be augmented to secure 2 500 gal. per min., usually by the addition of crane lines and water cranes.

11.—Transmission of the approved plans to the Department of Construction and Forestry for construction.

12.—Following up all schemes of every nature not entirely completed.

13.—Special investigation of doubtful sources of supply.

14.—Preparation of priority construction programs in order to use the small amount of material available to the best advantage.

15.—Inspections to aid in expediting construction and procurement of construction material from America.

Inspections and Surveys

Inspections made during April and May, 1918, showed that on the First Line of Communication the intermediate water stations had several months' reserve capacity for the transportation program of 1 500 000 men. Therefore, the supplies for the new yards and terminals were given the priority for immediate study. The facilities to suit each scheme were then indicated on the large scale maps prepared by the Engineer of Construction and forwarded to the Department of Construction and Forestry for action.

Plans and detailed information for all existing water stations on the First, Second, and Third Lines of Communication (Fig. 2) were requested May 1, 1918, from the P.-O., P. L. M., and Est Railways. These data were translated and compiled, together with computed drainage areas from contour maps and estimates of capacities per minute of local pipe lines.

These studies revealed the fact that several points which would have been suitable as to spacing, did not have sufficient supplies for future requirements. The deficient stations were pointed out in conference with the Water Supply Section of the Department of Construction and Forestry, with requests for stream measurements in July, August, and September. The Water Supply Engineer of the Department of Military Engineering and Engineering Supplies was asked to furnish data on rainfall and run-off in France for estimating probable yields.

Up to this time, nothing had been done to determine dry-weather flow. Where water was required for railroad use such information was of the greatest importance, as the maximum traffic was expected to coincide with the dry season. The following statement taken from a report of the Assistant Engineer of Water Supply very ably illustrated some of the difficulties:

"The line from Perigueux to Argenton-sur-Creuse may be taken as a typical section of the lines of communication on which doubtful sources of supply existed [see Fig. 2]. The drainage areas of streams adjacent to the French roadside water stations were drawn and studied in connection with the existing supplies and the locations of stations to be used as water stops for American trains were determined to use the best existing facilities as far as possible and conform to the desired spacing of from 20 to 25 miles. The water stations chosen were Perigueux, Thiviers, Nexon, Ambazac, La Souterraine, and water station for heavy south-bound trains climbing the grade from Argenton to La Souterraine. The average annual precipitation at these different water stations varied from 750 mm. at Celon to over 1 000 mm. at Ambazac, and Thiviers. La Souterraine, between Celon and Ambazac, and Nexon, half way between Ambazac and Thiviers, had annual rainfalls at 850 mm. Figures on the distribution of the rainfalls throughout the year were not

obtained, but observation through the summer demonstrated that the rainfall during the dry period was much greater in the mountainous region of Ambazac than it was in the less mountainous areas near the other stations.

"Storage or new sources were, therefore, necessary to supply deficiencies at La Souterraine, Nexon and Celon, although the drainage areas at these points were nearly as large as that giving an ample supply at Ambazac. Accordingly, it was planned to build a dam on the stream below Celon to store water for the dry period of 1919, and to lay a pipe line to the existing dam of Etang du Chi, near La Souterraine, which had sufficient capacity and elevation to furnish the additional water requirements by gravity."

Water supply problems similar to the foregoing were found on all points of the line, and attention has been drawn to this one simply for the purpose of illustrating conditions, rather than to convey the impression that it was a particularly bad territory.

TABLE 1.—WATER FACILITIES REQUIRED BETWEEN ST. NAZAIRE AND NEUFCHATEAU (*via* CERCY LA TOUR), FRANCE.

Stations and intervals (miles).	Mile posts.	Gallons per day.
St. Nazaire (6)	0	Some facilities
Montoir (14)	6	427 000
St. Etienne de Montluc	20	135 000
Nantes	39.7	....
St. Luce	....	160 000
Oudon (30)	49	180 000
La Possonniere (24)	79	180 000
La Menitre (16)	103	180 000
Saumur (24)	119	587 000
Langeais (28)	143	260 000
Dierre (22)	171	260 000
St. Aignan-Noyers (13)	193	260 000
Gievres (26)	206	817 000
Foecy (24)	232	300 000
Savigny en Septaine (26)	256	370 000
Le Guetin	282	370 000
Cercy la Tour (19)	321	1 173 000
Luzy (24)	340	700 000
Marmagne sur Creusot (33)	364	700 000
Santenay (29)	397	700 000
Vougeot (12)	426	700 000
Is sur Tille (19)	438	1 561 000
Villeguisien (23)	457	760 000
Avrecourt (Chal.-Neuf) (19)	480	360 000
Bourmont (13)	499	360 000
Neufchateau	512	427 000
Vitrey-Vernoise (Chal.-Epinal) (21)	481	360 000
Conflans-Varigney (23)	502	360 000
Xertigny (12)	525	360 000
Epinal	537	427 000

NUMBER OF STATIONS PROJECTED

On the First Line of Communication (Fig. 1) from St. Nazaire and Bordeaux to the rail-heads there were ninety-two French water stations. All these stations were studied carefully and the results tabulated on data sheets similar to those already described. (See Fig. 2.)

The necessity for an early start being apparent, a preliminary list of forty-one water stations was sent to the Department of Construction and Forestry on April 19, 1918 (see Tables 1 and 2). This was some time before it was possible to secure and work out any detailed information, and after only one inspection trip had been made. Later studies caused only minor changes in

six of these preliminary locations. The estimated quantities also varied but slightly.

The lines from Brest to Tours *via* Le Mans; from Rochefort and La Rochelle to Saumur and the so-called Second and Third Lines of Communication had ninety-two stations which were also tabulated and studied on receipt of the necessary information. (See Figs. 1 and 2.) Thus, there were 184 water stations covering 1 760 miles, which were studied in connection with the program in effect as early as April, 1918.

TABLE 2.—WATER FACILITIES REQUIRED BETWEEN BASSENS AND PONT VERTE, FRANCE.

Stations and intervals (miles).	Mile posts (from Bordeaux).	Gallons per day.
Bassens (10)	2	470 000
St. Sulpice (18)	12	150 000
Coutras (26)	30	250 000
Mussidan (24)	56	250 000
Perigueux (23)	80	570 000
Thiviers (25)	103	390 000
Nexon (25)	128	390 000
Ambazac (25)	153	390 000
La Souterraine (23)	178	260 000
Celon (25)	201	260 000
Chateauroux (32)	226	390 000
St. Florent	258	260 000

By November 11, 1918, 260 stations had been tabulated and studied in detail as to their availability for American transportation. Of these, 105 were selected for reserve or entirely new stations to fit the transportation program. By that time plans had been prepared for all the distributing systems, most of which had been approved by the French and transmitted to the Department of Construction and Forestry, while the remainder were in the hands of the Franco-American Mission (Fig. 2).

Thus, at the time of the Armistice, 105 plans had been completed for 1 900 miles of railroad; studies were well under way for 1 000 miles more; and data had been assembled and plans made for 100 water stations on the 1 500 miles of railroad which would have been required to advance the rail-heads to the Rhine.

SPACING OF WATER STATIONS

A number of factors must be considered in spacing railroad water stations: First, the rate of evaporation at different locomotive speeds must be known; second, the capacity of the tenders; third, the profile to be traversed; and, fourth, the method or scheme of operating trains. The last factor can only be anticipated in view of many years of past experience.

The tanks of the American locomotives held 4 900 to 5 300 gal. of water. The rate of consumption was 70 to 120 gal. per mile with a locomotive having its full tonnage rating. The variation was caused by the rise and fall in the grade line. A locomotive running on a level track uses more water per mile than one with the same load on a profile having many changes from adverse

to favoring grades, where the engine works at full throttle ascending, but can coast down the favoring grades without using steam. The maximum rate of consumption takes place on an adverse maximum grade involving possible engine delays, so that it would not be safe to have less than 1 000 gal. of water in the tender. For these reasons, a maximum span of 30 miles was adopted with an average of 20 to 25 miles. The small French water stations were located between these points and could be used in an emergency.

American Locomotive Used

The principal American locomotive was similar to one class of French locomotives. The statistics of the engine were about as follows:

Cylinders	21 by 28 in.
Steam pressure	190 lb.
Diameter of driving wheels	56 in.
Wheel-base of driving wheels	15 ft. 6 in.
Grate area	32.7 sq. ft.
Tractive power	35 600 lb.
Weight, in working order	166 400 lb.

A total of 3 490 standard-gauge Consolidation locomotives were purchased for overseas use. This is more than twice the total number operated by the Chicago and Northwestern Railway Company with 8 500 miles of line. Not all of them were delivered, however.

In all, 1 303 Consolidation locomotives were shipped for use by the A. E. F. In addition, 332 locomotives were en route to France or on docks in the United States awaiting shipment at the time of the Armistice. Also, 30 saddle-back switching locomotives were in use. Hence the undertaking to supply them with water was not a small one. On 1% grade, an engine would haul about 1 450 tons at the speed required under the circumstances.

General Plans

The roadside water stations having been selected, the French station plans were copied and the new facilities shown on them. These stations were of two general types (Fig. 4), one for trains with single engines, the other for helper engine districts.

The standard distributing equipment consisted of a 50 000-gal. wooden tank on a wooden tower located as close to a No. 11 Sheffield 10-in. water crane as possible. The pipe line from the tank to the crane was of 10-in. threaded pipe with couplings. Existing French pipe lines were used, when available, by the addition of other lines to increase the capacity to that of the 10-in. line. Water cranes were placed, when possible, between the main track and passing sidings so that engines could take water without blocking the main line. They were designed to furnish water to American locomotives during 5-min. stops (Fig. 4).

At a few points, such as the Perigueux Engine Terminal, high ground was available for the concrete reservoirs, but this was rare. The prime requisites

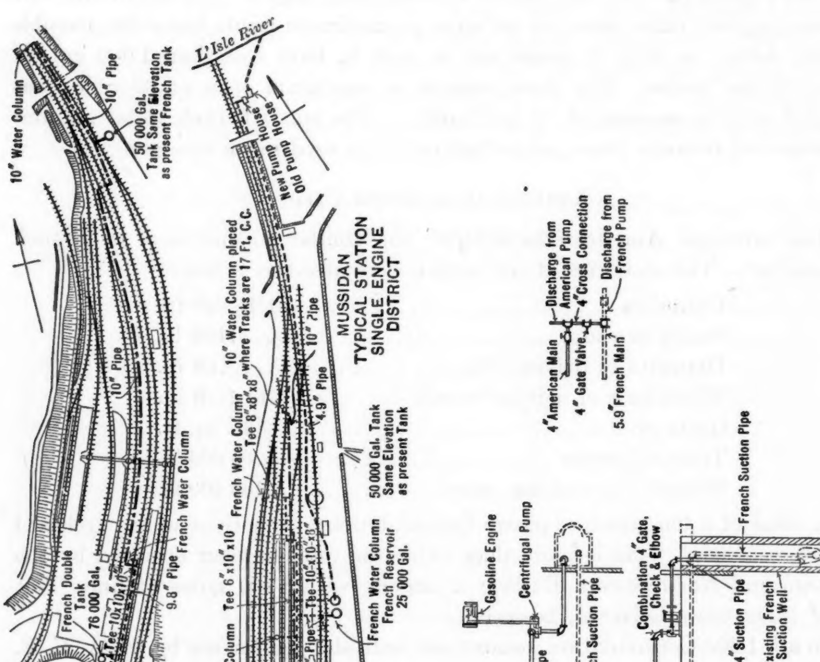


FIG. 4.—TYPICAL REINFORCED ROADSIDE WATER STATIONS AND STANDARD EQUIPMENT, TRANSPORTATION CORPS, A. E. F.

for watering a locomotive quickly are pipe lines of large volume and water at comparatively low heads, to avoid valve and pipe-line troubles from water ram due to sudden closing of large valves.

The quantity of water needed at each of these stations was calculated from the average daily tonnages necessary to supply the Army. It is obvious that military situations might have changed the whole conception of the matter and have required any one of the lines to be used at its full capacity. This was taken as 3 to 4 trains per hour in each direction for a 24-hour day.

At engine terminals, 5 000 gal. per engine entering was used. As the terminal spacings were the same as for the roadside stations, it was assumed that each engine would take 3 500 gal. of water when entering and 1 500 gal. would be needed for losses while standing idle. An allowance of 10 000 gal. per day was made for each switch engine and from 30 000 to 90 000 gal. per day for power plants, boiler washing, and miscellaneous uses.

Fortunately, the quality of water did not complicate the situation to any extent. Most of the water from the coast to the northeast part of France is what is called "light". It is not very hard until the Vosges territory is reached. At the sea coast, in 1918, considerable trouble was experienced with foaming. During the summer months, the reservoirs were pumped down so that seepage changed from the landward side to the seaward side. The salt water caused the trouble.

Organization Difficulties

The actual working history of an organization is usually dry reading. However, as similar situations may occur in any campaign, it seems advisable to give a brief outline of organization troubles that added immensely to the work without in themselves producing any water stations; and it was water stations that were required.

On January 5, 1918, preliminary plans, including water requirements at several engine terminals, were sent to the Chief Engineer Officer, Lines of Communication (later Department of Construction and Forestry). On April 19, 1918, a set of tables of water requirements (Tables 1 and 2) was sent by letter to the Department of Construction and Forestry. On May 6, 1918, a list of seventeen engine terminals (Table 3) was sent to the same destination. It gave estimates of requirements for 500 000, 1 000 000, and 1 500 000 men. On May 25, 1918, a report was sent dealing with conditions between St. Nazaire and Saumur, pointing out the advisability of investigating ground-water conditions at Saumur, an engine terminal and very important point. By memorandum of June 6, 1918, notes of a trip between Perigueux and Chateauroux were sent to the same officer. No reply having been received prior to June 19, 1918, two letters were sent, requesting ground-water surveys at Saumur and St. Luce and stream measurements in different localities.

To add to the confusion, the P.-O. Railroad officials, on June 30, 1918, presented detailed descriptions of what they wanted done to nine water stations and requested immediate action. The available material in stock in the Engineer Depots was determined at a conference with the Department of Construction and Forestry and a modified plan produced to satisfy the French officials.

FIG. 4.—TYPICAL REINFORCED ROADSIDE WATER STATIONS AND STANDARD EQUIPMENT, TRANSPORTATION CORPS, A. E. F.

This plan was furnished to the officials of the P.-O. Railroad on July 6, 1918, and the Department of Construction and Forestry was requested to furnish the equipment by letter of July 4, 1918.

TABLE 3.—WATER INSTALLATION NEEDED AT VARIOUS POINTS IN THE ORDER NAMED.

Yard and terminal locations.	REQUIREMENTS, IN GALLONS PER DAY.		
	For 500 000 men.	For 1 000 000 men.	For 1 500 000 men.
St. Nazaire	50 000	100 000	150 000
Montoir	142 000	284 000	427 000
St. Luce	17 000	34 000	50 000
Saumur	176 000	352 000	527 000
Gievres	272 000	544 000	817 000
Cercy la Tour	391 000	782 000	1 173 000
Is sur Tille	520 000	1 040 000	1 561 000
Liffol le Grand	142 000	284 000	427 000
Villiers le Sec	....	....	750 000
Bassens	157 000	314 000	470 000
St. Sulpice	50 000	100 000	150 000
Perigueux	190 000	380 000	570 000
Ambazac	130 000	260 000	390 000
Nexon	130 000	260 000	390 000
Chateauroux	130 000	260 000	390 000
La Rochelle	125 000	125 000	125 000
Aigrefeuille	50 000	50 000	50 000

These negotiations with the officials of the P.-O. Railroad were but a forerunner of numerous conferences and illustrate the importance of having all information in the hands of any one entering such discussions. A letter was accordingly sent to the Department of Construction and Forestry on July 6, 1918, asking for joint reconnaissance surveys of proposed water stations by the representatives of the Engineer of Water Supply of the Transportation Service and Section Engineers of the Department of Construction and Forestry, with authority to agree on methods, etc. This letter was unanswered, and telephone inquiry only resulted in the information that the Section Engineers were too busy to pay any attention to these details.

The next move (July 17, 1918) was to appeal to the Commanding General, Service of Supply, for an adjudication. His decision is found in a memorandum of July 23, 1918, forwarded to the Department of Construction and Forestry, stating in part:

"It would appear that there can be no objection to permitting representatives of the Transportation Department working jointly with the Construction and Forestry Departments in investigating sources of water supply at various water stations. However, the responsibility for supplying an ample supply of water where needed, rests squarely on the D. C. & F. Any information of sources of water which the T. D. has, should be turned over to the D. C. & F. The importance of this matter is so great that immediate attention must be given to this problem." (See Fig. 3.)

No one was able to explain just how much of this information was to be turned over. It consisted in part of mental conceptions resulting from many years of experience in similar matters, and could not be made into a neat package for delivery.

During the negotiations, the Water Supply Section of the Department of Construction and Forestry secured a geologist to go over the line from Perigueux to Celon. He was accompanied by a representative of Engineer Water Supply of the Transportation Department. This was the beginning, and by September 10, 1918, the first complete reports of the proposed methods of developing doubtful sources of supply were received.

Engineers are prone to make curves to illustrate their meaning with the idea that busy men will get the point without effort. This was tried in order to relieve the busy Section Engineers of labor. It worked like the well-known Australian boomerang; the information was used by the Section Engineer of the Department of Construction and Forestry as an argument against the reinforcement of certain stations. The small French stations had to be used between engine terminals until all the reinforced stations in the district had been completed. Hence the small actual consumption at the selected roadside water stations was compared with the requirements for any day taken from the curves and used as an argument against the immediate necessity of proceeding with construction.

Difficulties in Securing Material

The rapid increase in the size of the Army and the military activities due to the St. Mihiel Drive in September, 1918, produced such a density of traffic as to bring out forcibly the weak points. The roadside water stations had not been reinforced to deliver water rapidly, and all trains were stopping at the small French water stations to take water. Sometimes they obstructed the main track for 20 min., thus automatically limiting train movement to 2 trains per hour. Such conditions finally brought the problem of railroad water supply into the spotlight, and this resulted in accelerated construction which thereafter advanced as fast as material was received.

Lack of suitable material was the most serious factor in delaying construction. Requisitions had been made for material, but the shipping tonnage alloted the Transportation Department was so small that it was deemed necessary at first to utilize all of it for locomotives, cars, rails, etc. When the new Department of Engineer of Water Supply was formed, in April, 1918, it was discovered that no material for water service had arrived and none had been called for by the Department of Construction and Forestry during the previous two months, when the work had been under its jurisdiction. Consequently, the first shipment was requested on April 18, 1918, as a part of the tonnage quota of the Director General of Transportation. The same was true of May and June.

The Engineer Water Supply explored every possible channel for procuring tanks in Europe, but without success. The only ones found were several 25 000-gal. wooden tanks made by the construction forces at Gievres. These were excellent, but construction of them had been discontinued. Thus, it was necessary to depend entirely on shipments from the United States. When notice of the first shipment of tanks was received, a list of the points where they were most urgently needed was given to the Department of Con-

struction and Forestry. This was done from time to time, but by October 17, 1918, it became apparent that only the tanks on the original requisition had arrived. (For that matter these were all that ever did reach France.) A hurry-up movement was then instituted to get shipments of tanks for the proposed drive to the Rhine, but this was stopped by the Armistice.

In conference with the officials of the P. L. M. Railway, it was found that they could use German war prisoners to build reinforced concrete tanks for American transportation by contract. This scheme was carried into effect at three different points before the cessation of hostilities, but, even then, American water cranes had to be used, for not even one was obtainable in France.

Paper Work and Construction

Quoting again from an official report:

"However, as the organization stood, a great deal of the work was only done on paper with a comparatively small accomplishment in actual construction. An excellent example of this may be seen in the case of the roadside water station reinforcement proposed at Thiviers. In all there were 46 letters, telegrams, and conferences extending over a period of six months, with which the Engineer of Water Supply was concerned, on this one project. And at the time the Armistice was signed, the construction had not started, although a shipment of 6-in. supply pipe had arrived a short time before." (See Fig. 3.)

By the careful assignment of available material to the new engine terminals, yards, and weakest roadside stations, it was possible for the American Lines of Communication to carry one-third of the total tonnage needed to supply 4 000 000 men. In November, 1918, water facilities had been installed sufficient for 1 300 000 men, which was nearly the size of the Army contemplated in July, 1918, for the campaign in 1919.

When the Armistice was signed, eighty of the projects were canceled, the remainder being used for the different movements needed for the return of troops to the base ports, the maximum traffic being reached about the middle of December, 1918.

In addition to the principal duties in connection with train movement the Engineer Water Supply, Transportation Department, was called on for various other odd jobs. Among these were fire protection at Bassens docks, water facilities on forestry sidings, supplying drinking water to train crews and to troops in transit, aid in the rehabilitation of the Lines of Communication through the fighting area into Germany after the Armistice was signed, and aid in operating the lines taken over after the St. Mihiel Drive. His advice and assistance in securing all kinds of approvals from the French were necessary before they could be carried out.

One of the problems that was carried to a conclusion was that of mending the damaged water supply at Couflans-Jarny (on the front north of Toul) on the section of railroad near the Luxembourg border, operated by the Americans. This terminal had been badly damaged by American artillery fire. The large distributing mains had been broken in six different places and the pumping machinery carried away by the Germans. The mending

of the breaks and the installation of a steam pumping plant was carried out under the direction of the Department of Engineer of Water Supply and a report submitted recommending what was needed at other water stations on the section under American control.

At the time the Armistice was signed, the writer was investigating a very mysterious cause of foaming on the lines of the Woevre (near Thiaucourt). This was on a line that had been taken over after the St. Mihiel Drive Engines foamed on Wednesdays and Thursdays. A voyage of inspection of several miles up the creek from which the water was taken disclosed a village with a large public laundry. The inhabitants washed their lingerie on Mondays and Tuesdays, the soda and soap reached the engines on Wednesdays and Thursdays; hence, the disturbance. It is the only case on record during years of experience, in which engine foaming conformed to the calendar. As it was impossible, with due regard to the *entente cordiale,* to sentence the population to wearing soiled shirts, nothing was done about it, except to protest.

Conclusions

The lessons to be drawn from all these experiences may be summarized briefly as follows:

The Engineer of Water Supply, Transportation Department, should have primary control of certain fundamentals with sufficient personnel to carry matters to a conclusion:

1.—All investigations necessary to determine the sources of water supply for transportation.

2.—Design of supply systems to the extent of specifying capacities of pipe lines to be installed and general location plans.

3.—Control of essentially railroad water service material, such as tanks, water cranes, valves, piping, engines, and pumps.

4.—Installation of water tanks, water cranes, and connections for roadside water stations.

5.—Control of a maintenance force for their upkeep. When accidents happen, the enginemen must report it by wire to the dispatcher so that he can change his train movements accordingly. The repair forces should be within easy reach of this source of information.

There is no essential difference between engines, cars, coal, and water as parts of a transportation scheme, and they should be under the same general head. To assemble engines and cars is not far different from erecting water stations. It is believed that the supply of equipment for indefinite problems is best handled from one large depot and is, therefore, rightly under the Department of Construction and Forestry. This includes pumps and engines which can more readily be selected from a large stock.

The main object in work of this sort is to preserve the continuity of all operation. This has been accomplished under civilian control by the organization of "all line gangs". These are under the control of one official who can

move them about into different divisional jurisdictions. Similar forces, completely equipped, could have simplified matters in France. The chief obstacle to be avoided in transportation is delay. As a chain is only as strong as its weakest link, so local delays may interfere with train movements perhaps hundreds of miles away.

It would not seem difficult under military organization to accomplish this by the creation of a separate unit. This unit would have absolute control over its own activities from the starting point, or investigation, to the conclusion—a completely equipped water station with sufficient supply to enable it to deliver water to locomotives at points best suited to the expediting of traffic.

PROBABILITY OF FLOOD FLOWS

By F. G. Switzer,* Assoc. M. Am. Soc. C. E.

Synopsis

Frequency curves may be applied to flood data in determining flood flow probabilities. The results are shown in the paper, also the relation between floods and drainage areas, for floods of equal probability.

The problem of the prediction of rainfall should be subject to analysis. Some progress has been made in the long-distance prediction of the coming and duration of rainfall. Thus far, little has been done in predicting at long range the intensity of rainfall to be expected. In the absence of information regarding the factors which govern the intensity of rainfall, consider the idea from a different angle. The intensity of rainfall varies from storm to storm. There are many rains of low intensity and few of high intensity. The higher the intensity, the less becomes the number of occurrences. This at once suggests that one of the probability curves† may be used to fit such data. A valuable discussion of frequency and probability curves has been given‡ by H. Alden Foster, Assoc. M. Am. Soc. C. E. Mr. Foster applies these curves to the determination of the probability of annual stream flow. It is the purpose of this paper to apply the same method to the determination of the probability of flood flow.

Knowing something of the past floods on a river, and their frequency, an extension of the frequency curve at values of low probability is the desired end. The first requirement is a record of the past performance of the river. Of course, the longer the record the better. The next requirement is the proper use of these data. The writer has given considerable study to this phase of the problem. In one case, the daily discharge record of the Coosa River, Alabama, for a period of twenty-eight years was used. More than 10 000 separate values were thus obtained. Again, the annual peak flows for the same period were used.

These two sets of data gave widely different results. Different water-years were tried. This river is subject to floods during a considerable part of the year; hence, the choice of water-year may radically change the data.

Note.—Written discussion on this paper will be closed in **August, 1927.** When finally closed the paper, with discussion in full, will be published in *Transactions*.

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† A rather complete discussion of frequency curves, from which the probability curve may be derived, is given in "Frequency Curves and Correlation", by W. Palin Elderton.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 142 *et seq.*

For instance, taking the year as from October 1, it was found that two or more high flows sometimes occurred in one year with no particularly high flow in the preceding or following year. By taking the water-year from January 1 a different set of flood flows was obtained.

The frequency curves were radically different. In the first case there were fewer excessive flows than in the second because several excessive flows sometimes came in one water-year, of which only the greatest was used. The lesser flows omitted were in many instances greater than the maximum flood in the preceding or following years. It was finally decided to pay no attention to the date when any flood occurred. A list was made containing the maximum 24-hour flood for every storm that caused a peak flow in excess of an arbitrary value. These data were used to obtain a frequency curve, from which a probability curve was obtained. A comparison of the results is shown in Table 1.

TABLE 1.—FLOW TO BE EQUALLED OR EXCEEDED ON THE AVERAGE OF ONCE IN THE TIME PERIOD, COOSA RIVER, ALABAMA.

Time period, in years.	From annual peaks October 1-October 1, in second-feet.	From daily record, in second-feet.	From flood flows, in second-feet.
10	131 000	166 000	168 000
100	179 000	222 000	226 000
1 000	236 000	278 000	280 000

The value of the arbitrary flow below which no floods were included has apparently little or no effect on the result, provided it is set low enough to permit the inclusion of a reasonable number of floods. It should be noticed how closely the daily record results and the flood flow results check.

The greatest recorded flow in the Coosa River at the site for which data were taken has been 190 000 sec-ft. The frequency curve indicates that this flow may be equalled or exceeded on the average of once in 29 years. There is nothing to indicate whether such flows will occur in successive years. The data merely indicate that the average interval between floods of equal or greater magnitude is 29 years.

In another study, it was desired to learn something of the effect of completeness of record. To this end the flood data of another river for a period of 10 years were used. A second computation was made in which the two highest floods were omitted. The results are compared in Table 2.

TABLE 2.—TWENTY-FOUR-HOUR FLOOD TO BE EQUALLED OR EXCEEDED ON THE AVERAGE OF ONCE IN THE TIME PERIOD.

Time period, in years.	Flow, in second-feet, complete data.	Flow, in second-feet, two floods omitted.
10	68 000	57 000
100	100 000	72 000
1 000	132 000	82 000

It will be noted that the percentage difference between the two sets of data is increasingly large at higher flows. This comparison is not for determining the effect of length of record. With a longer record, the greater number of high flows is accompanied by a much greater number of lower flows. However, it is interesting to find such a difference due to the omission of 2 points from a curve determined by 294 points. The frequency curve equations are determined by the moment method. The axis of moments is at a low flow. Hence, the moments of the big flows are important because of their magnitude and moment arm. This gives greater weight to the large flows, and, consequently, the equation of the curve obtained will more closely fit the higher flows than the lower. This is desirable for flood study.

The records of the three following rivers in Alabama were studied: Tallapoosa River, at Sturdivant; Coosa River, at Lock 18; and, Tennessee River, at Florence. Pertinent data are given in Table 3.

TABLE 3.—DATA OF ALABAMA RIVERS STUDIED.

	Tallapoosa.	Coosa.	Tennessee.
Length of record, in years	20	12.5	51
Minimum flow considered, in second-feet	5 000	24 000	23 100
Number of storms, in record	294	91	794
Drainage area, in square miles	2 460	10 165	30 800

For each river the flow peaks were arranged in numerical order. The peak 24-hour flow was used except for the Coosa River where the maximum gauge height was available. These data were used for computing the constants of the frequency curve according to Elderton's method. For plotting the probability curves, the coefficients as given by Mr. Foster were used. For some reason, probably an arithmetical error in computation which could not be found, the curve for the Coosa River thus obtained did not fit the data. For this case the frequency curve was integrated by expanding the exponential terms into a series which was convergent. Some trouble was encountered before the resultant series was made to be convergent. For purposes of reference, the constants were found for the frequency curves, as given in Table 4.

TABLE 4.—CONSTANTS OF FREQUENCY CURVES, ALABAMA RIVERS.

	Tallapoosa.	Coosa.	Tennessee.
Type	III	I	III
Coefficient of variation	0.755	0.514	0.692
Coefficient of skew	2.232	1.692	1.480

The three probability curves, with the data from which they were obtained, are shown in Figs. 1, 2, and 3. These data are plotted on probability cross-section paper.

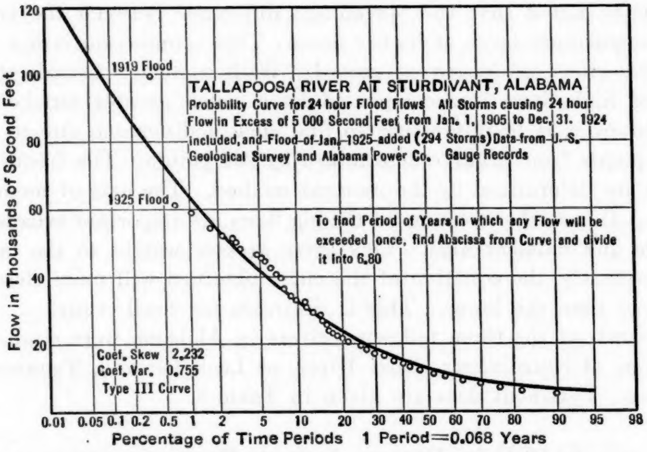

FIG 1.—FLOOD PROBABILITY CURVE FOR TALLAPOOSA RIVER, ALABAMA.

Since the number of storms is different from the number of years, the time scale is the percentage of the time periods. One time period is the number of years of record divided by the number of storms found. To find the period of years in which, on the average, any given flow will be equalled or exceeded once, the percentage of the time periods, expressed as a decimal, should be divided into the length of the time period.

The curves of Figs. 1, 2, and 3 have been replotted in order to show the flood to be equalled or exceeded on the average of once in any period of years. These curves are shown in Fig. 4 (*a*). In Fig. 4 (*b*) are shown the same curves replotted by changing the ordinates to show the discharge per square mile of drainage area. Again, in Fig. 4 (*c*), the same curves are replotted with ordinates of discharge divided by drainage area raised to the 0.6 power. It will be noted how the three curves cross each other and so

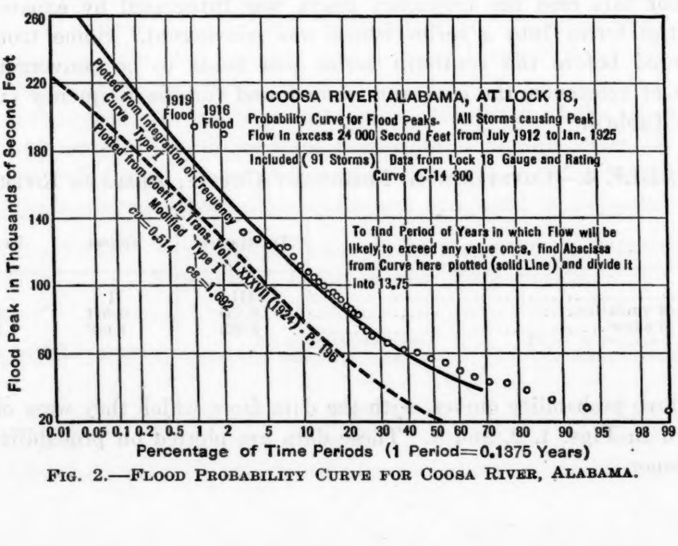

FIG. 2.—FLOOD PROBABILITY CURVE FOR COOSA RIVER, ALABAMA.

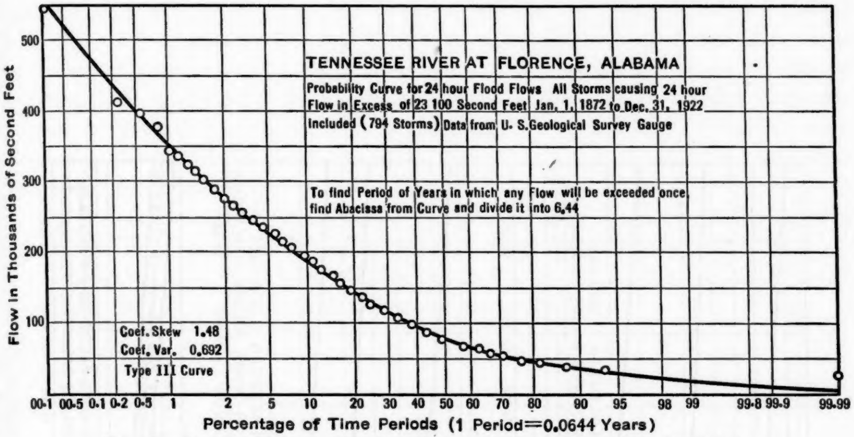

FIG. 3.—FLOOD PROBABILITY CURVE FOR TENNESSEE RIVER, ALABAMA.

nearly coincide. This would seem to indicate that floods of equal probability are proportional to this function of the drainage area. Using the symbols of C. S. Jarvis, M. Am. Soc. C. E.,*

$$Q = c\ M^{0.6}$$

in which, Q = the discharge, in second-feet, and M = the drainage area, in square miles. Values of c for floods which will be equalled or exceeded on the average of once in 10 and 1 000 years are 700 and 1 300, respectively. This formula is shown only for the purpose of inducing additional work on the subject. With the limited number of data, it can now have only local application.

TABLE 5.—PROBABILITY OF GREATEST RECORDED FLOODS.

River.	Greatest flood, in second-feet.	Average interval, in years, between floods of equal or greater magnitude.
Tallapoosa	99 000	90
Coosa	190 000	29
Tennessee	540 000	500

The particular merit which may be claimed for the method herein given is that the resulting formula will be obtained from data of equal probability. The choice of constant can then be based on a semblance of economies, inasmuch as it is possible to start with a span of years as the average life of any construction and design for a flood that will be equalled or exceeded on the average of once in that period. Another way of attacking the problem would be to find the cost of structures designed for different flood dis-

* "Flood Flow Characteristics," *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 993, *et seq.*

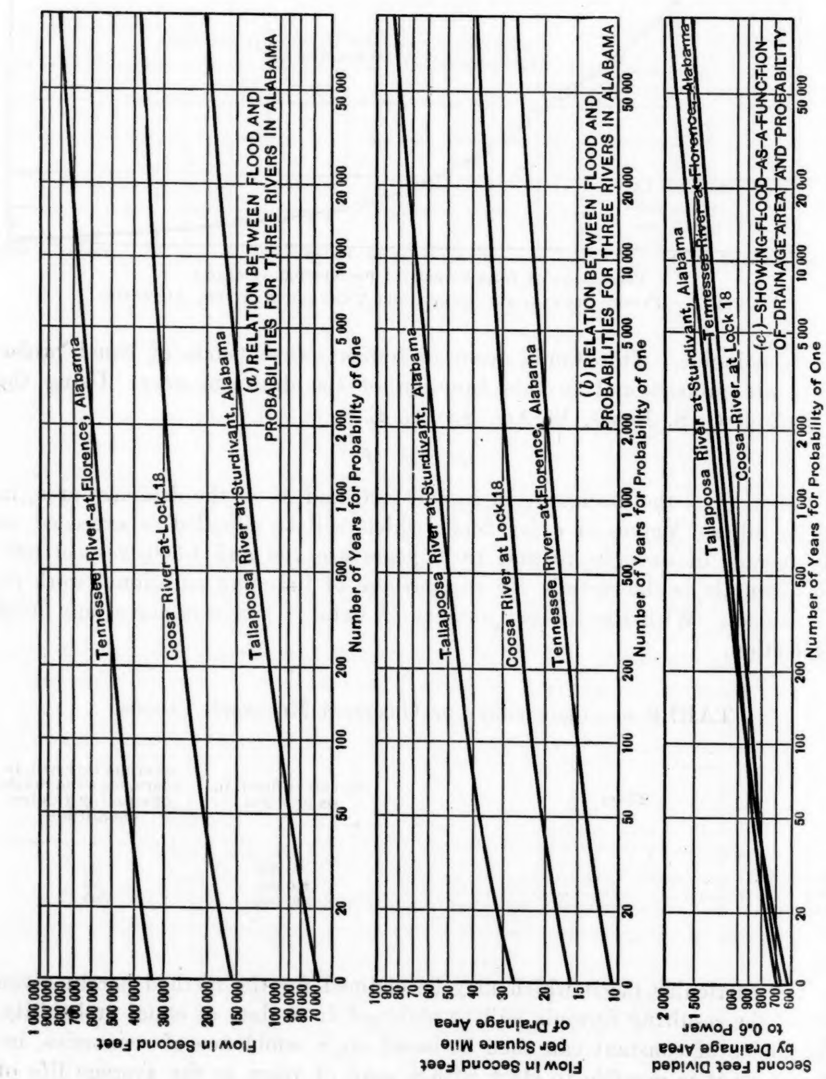
(a) RELATION BETWEEN FLOOD AND PROBABILITIES FOR THREE RIVERS IN ALABAMA
Tennessee River at Florence, Alabama
Coosa River at Lock 18
Tallapoosa River at Sturdivant, Alabama
Flow in Second Feet
Number of Years for Probability of One
(b) RELATION BETWEEN FLOOD AND PROBABILITIES FOR THREE RIVERS IN ALABAMA
Flow in Second Feet per Square Mile of Drainage Area
(c)—SHOWING FLOOD AS A FUNCTION OF DRAINAGE AREA AND PROBABILITY
Second Feet Divided by Drainage Area to 0.6 Power

FIG. 4.

charges and divide this by the average number of years in the interval between equal or greater floods. That structure which showed the lowest cost per year of service would then be adopted. In calculating costs, items of interest, and contingent flood damages due to the failure of the structure should be included in some way. The amount of work involved in such study has prevented the extension of this method. It is hoped that more data will become available from time to time.

To indicate the non-homogeneous nature of data used in obtaining any curve drawn through the greatest recorded floods, Table 5 is presented.

The writer wishes to acknowledge his indebtedness to the Alabama Power Company for the use of the data herein presented.

AERIAL SURVEYS FOR CITY PLANNING

Discussion*

By Gerard H. Matthes, M. Am. Soc. C. E.†

Gerard H. Matthes,‡ M. Am. Soc. C. E. (by letter).§—Among the many valuable points brought out in the discussion of this paper a number merit particular attention.

Mr. Ripley‖ points out the desirability of a county retaining possession of the negatives of an aerial survey made for it by contract. The disposal of negatives has been a much mooted question ever since aerial surveying expanded into an industry. Retention of aerial negatives by the contractor is predicated on the well-established custom of a photographer retaining ownership of the negatives of photographs which he makes for his client. Some aerial mapping companies regard the negatives as the property of the client but to be retained by them for subsequent use subject to the orders and wishes of the client.

Aerial mapping contracts sometimes specify reimbursement to the client of part of the profits derived by the contractor in selling copies of maps or pictures from the client's negatives. In the case of municipalities this practice is not to be recommended. Delivery of the negatives would seem an essential to a complete discharge of the terms of any public contract. A precedent for this has been established by the contract for the aerial map of New York City let by the Board of Estimate and Apportionment, which called for the delivery to the city of all negatives, including the original negatives as well as the large copy negatives from which the individual map sections were made. The wisdom of this procedure appears to be confirmed by the action taken by Erie County.

On the subject of ground control for aerial surveys for city planning purposes, the point raised by Mr. Olmsted¶ is well taken, namely, that it will often be advantageous for a city to establish a geodetically precise horizontal control once and for all, to serve as a base for future surveys, including those for cadastral and construction purposes. However, as few city engineers are in position to undertake such geodetic surveys for lack of specially trained personnel and special instrumental equipment, such work is usually best done by contracting with properly equipped and qualified firms engaged in such

* Discussion on the paper by Gerard H. Matthes, M. Am. Soc. C. E., continued from January, 1927, *Proceedings*.

† Author's closure.

‡ Cons. Engr., New York, N. Y.

§ Received by the Secretary, March 1, 1927.

‖ *Proceedings*, Am. Soc. C. E., January, 1927, Papers and Discussions, p. 94.

¶ *Loc. cit.*, December, 1926, Papers and Discussions, p. 2025.

business, or else, by enlisting the co-operation of the U. S. Coast and Geodetic Survey. This Bureau, in the past, has performed valuable service in extending geodetic surveys over certain city areas. Unfortunately, the limited personnel available for this work makes it impracticable for the Coast and Geodetic Survey to undertake co-operative surveys of this kind to any extent.

The discussions have emphasized the practical value of oblique aerial photographs and controlled mosaics, two phases of aerial surveying that are not receiving the recognition that is due them on the part of many engineers, who are prone to class them as mere pictures and picture maps. The fact is that already these two forms of aerial photography have been used effectively in a surprisingly large number of instances where formerly line maps would have been demanded. For certain purposes they have proven so satisfactory (and better than line maps) that the demand for them on the part of engineers and others is constantly increasing. This is especially true in the case of surveys for city planning.

WATER-RATIO SPECIFICATION FOR CONCRETE

Discussion*

By Thaddeus Merriman, M. Am. Soc. C. E.

Thaddeus Merriman,† M. Am. Soc. C. E.—The authors are to be commended for their frank recognition of the fact that compressive strength bears no relation to either the permanence or the durability of concrete. They have presented one of the best practical statements on the design of concrete mixes that has yet been prepared.

Quite aside from "the design of concrete mixtures" there is another aspect of the problem of concrete, which demands attention. The wheels of progress seem to have revolved to that point on the road to progress where reasonable agreement in regard to the matters of water-ratio and proportions is indicated; but this point does not mark the journey's end. Because agreement has been reached on the obvious things, which are visible to the eye, it does not follow that the problem has been completely solved. From these gross physical manifestations the inquiry must go forward to a consideration of the conditions under which the cement reactions occur and seek to ascertain how it will be possible to secure both an optimum of concrete quality and a maximum of permanence and durability.

A practical demonstration will serve to illustrate differences in quality of product. Mix a sample of cement with 43% of water by weight. Pour one-half this mixture into one bottle, the other half into another. Break the first bottle at the end of 24 hours and allow the sample to remain in the air. The second bottle should be broken when 21 days old and the sample thereafter kept in the air. These two samples will speak for themselves. The first will be soft, porous, and chalky, while the second will be hard, dense, and of rock-like structure. Made from the same cement, they will look like two entirely different products. The chemical reactions in the one sample will evidently have been of a very different order from those in the other.

The important question to be determined is that of learning why concrete in one place is good and strong while in another it disintegrates and shows little of either permanence or durability. The water-cement ratio seems to have some bearing on this matter, yet the day is not far distant when concrete will be designed on the basis of the proportionate relationships between (*a*) the cement and the water; and (*b*) the cement and the sand. Then, further, consideration must also be given to the environment in which the cement

* Discussion on the paper by F. R. McMillan, M. Am. Soc. C. E., and Stanton Walker, Assoc. M. Am. Soc. C. E., continued from March, 1927, *Proceedings*.

† Chf. Engr., Board of Water Supply, City of New York, New York, N. Y.

undergoes its hydrations. These conditions being under control, the user may then put in as much or as little stone as he desires.

Permanent concrete can hardly be designed on the basis of the water-cement ratio only. Of the many factors that go to determine the quality of concrete this ratio is only one, and it would be passing strange if this simple and obvious arithmetical fraction should constitute the last word in the act of concrete making. The factor of cement quality is not so easily to be submerged. Until the solution concentrations in which the cement reactions occur are held under control the nature of the hydration products cannot be predicted. These products, are those which impart to concrete its well-known but little understood characteristics. Until knowledge of cement and its behavior is vastly greater than it now is, but little more can be developed than is now known in regard to the mixture called concrete.

WATER-PROOF MASONRY DAMS

Discussion*

By Messrs. Nathan C. Johnson and F. W. Scheidenhelm

Nathan C. Johnson,† Assoc. M. Am. Soc. C. E.—Since the author's water-proofing membrane is proposed *per se* as a means to prevent uplift in dams by stopping the entrance of water into the concrete or masonry, is the situation not this: Assume a more or less hollow structure, of ample weight for stability, but made, say, of sheet iron so as to be impervious. There will be no uplift unless the structure, instead of being filled with water, is filled with air. There is, therefore, no uplift to be anticipated for any more or less hollow structure, but only for a structure from which water is excluded.

In a concrete dam there would be no uplift except through voids filled with air which are in a very small percentage, assuming that no water enters under the dam.

This viewpoint is taken in order to divorce the subject of uplift from that of concrete as a material. This latter is a separate subject, particularly as regards its abilities or disabilities for use in water-exposed structures.

The discussion of this paper, therefore, calls for division into (1) the question of entrance of water *versus* uplift in any water-retaining structure; (2) the actuality, magnitude, and distribution of forces exerted by uplift; and (3) concrete as a material suitable for water-bearing structures and the necessity, or lack of necessity, for the water-proofing concrete because of its inherent disabilities, or because of the result commonly produced in commercial work.

By its very nature concrete is never free from water, but it should not pass water in sensible quantities. If concrete is free from water, it is a disintegrating and more or less pulverulent mass. Concrete must have a normal water content to maintain its integrity; and any proposal to exclude water must be carefully evaluated before it can be pronounced advisable; and a clear differentiation should be made between the exclusion of water and the adoption of means to prevent percolation of water through the mass because, perhaps, of anticipation that a poor job will be had in the actual construction of the barrier.

As to concrete as it is commonly made and the prevention of percolating water, many means have been proposed to secure immunity from the evil of poor work, which is far too prevalent.

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* Discussion on the paper by W. Watters Pagon, M. Am. Soc. C. E., continued from March, 1927, *Proceedings*.

† Cons. Engr., New York, N. Y.

When Mr. Pagon proposes a water-proofing membrane for concrete dams, the speaker can see some values as a temporary preventive of first-hand leakage, but no preventive of either permanent proofness or of uplift.

In concrete construction, what is needed first of all is a more general appreciation as to the true nature of concrete, one that will be on a plane with the artistry of which draftsmen are capable in their designing. The draftsman's picture-section of concrete shows a uniform, homogeneous substance, while the product in the field is a highly variable and non-uniform substance and through this field-produced article, water passes in billions of gallons daily. On that rock, practice and theory are split asunder; and much of the confusion of thought that exists to-day arises from that one cause.

In conclusion, therefore, it seems to the speaker that Mr. Pagon should divorce the subject of uplift from the subject of water-proofing; that he should divorce the subject of concrete from the subject of uplift; and that his conception of a water-proof membrane in the face of the dam as preventing uplift is a misconception and unrelated to the subject of uplift *per se,* or to the designing of dams for either safety or economy.

F. W. SCHEIDENHELM,* M. AM. SOC. C. E.—The speaker finds himself thoroughly sympathetic with the desideratum, advocated by the author, of water-proof membranes to be located near and approximately parallel to the up-stream faces of masonry dams. In the reason for this sympathy, however, he differs somewhat from the author, who emphasizes probability of diminution in cross-section of a dam equipped with such a membrane and of consequent reduction in cost. The speaker, on the other hand, is attracted by the hope that such a membrane might constitute an important contribution toward the permanence of the masonry, especially if that be concrete.

In the paper much stress seems to be laid on the effect of hydrostatic uplift pressure, particularly within the body of a dam, in determining the cross-section and hence the economy of the structure. However, the speaker questions whether such interior uplift pressure is really so important as the paper implies. For instance, he has no direct personal knowledge of any failure of an upper portion of a masonry dam by overturning or sliding on a lower portion except where caused by ice thrust. Only a single well-authenticated case of failure of a dam even by overturning on its foundation bed other than due to ice pressure has come to his attention and that case is stated to have involved a dam only 20 to 30 ft. in height and of obviously too thin a cross-section.

It is true that uplift pressure in effect reduces the net weight of a dam or, more accurately, may reduce the load and hence frictional resistance on a possible plane of sliding, and that in this way uplift pressure may contribute to that most important form of failure of masonry dams, namely, failure by sliding. Even so, the uplift pressures that are most dangerous are those existing not within but under a dam, either between the masonry and the natural foundation bed or between approximately horizontal strata of the foundation bed itself. The author implies that he would utilize only such foundation beds as are "tight". However, if he can avoid seamy or porous foundation

* Cons. Engr. (Mead & Scheidenhelm), New York, N. Y.

beds he is singularly fortunate. More often the problem is one of recognizing a seamy or porous condition of foundation bed and of so designing the dam and its appurtenances as to make the resulting structure stable.

The failure of the dam at Austin, Pa., has been mentioned. Although uplift pressure may have been one factor, yet it was not necessarily the only factor and the failure was primarily one by sliding. Certain portions of a concrete cut-off wall, such as it was, remained embedded in the foundation rock—of partly disintegrated, argillaceous sandstone with clayey interbeddings. There must have been cracking or complete rupture of that concrete cut-off before any material uplift pressure could have been exerted under the body of the dam. Slender and inadequate as was the cross-section of the dam, the deficiencies in that respect were hardly so serious as those involved in the provision against sliding. The dam might have slid even if there had been no uplift whatever.

There are various expedients for minimizing uplift pressures under a dam. First, there is the expedient of constructing a cut-off in which, as the author points out, it is possible to embody a membrane intended to be water-proof. (However, inasmuch as the concrete of a cut-off wall is not subject to alternate wetting and drying, or freezing and thawing, the speaker would not feel greatly concerned about such percolation through a cut-off as might occur in the absence of a membrane). The cut-off itself, whether it be of concrete or of sheet-piling, may be extended downward still farther by means of pressure grouting with cement, bituminous compounds, etc.

Against the contingency that the cut-off provision may not extend sufficiently deep, or for any other reason may not be absolute, there is possible a second line of defense, namely, a row of deep drain holes, relieved at the top by suitable means to tail-water pressure and extending downward as far as may be necessary. If, for instance, the base of a dam is 100 ft. wide, the line of such drains might well be 10 to 20 ft. down stream from the up-stream face of the dam and parallel thereto. Surely if these drain holes are spaced closely enough, they would be adequate to prevent any uplift pressure substantially in excess of that due to tail-water. Some might feel concerned that in course of time these drain holes would become clogged, but uplift pressure will presumably be as severe initially as at any time in the life of a dam and even a slight differential pressure will maintain an upward flow and hence relieve the pressure through the drain holes.

In short, it would seem that along the line of such a barrage of drain holes one may reasonably expect the intensity of uplift pressure to approximate that due to tail-water. Whether the maximum intensity of uplift pressure at the up-stream face of the dam be equal to 100% of that due to head-water, or whether it be some fraction thereof, such as two-thirds, is not, under such circumstances, of major consequence. Moreover, such uplift pressure as does exist can not obtain over the entire base of the dam; otherwise the dam would be afloat. Against encroachment of hydrostatic pressure from ground-water or of reservoir pressure coming in from the hillsides, one may similarly set up a defense by extending lines of drains down stream from and at right

angles to the main line of drains, such auxiliary lines, of course, being located close to the valley banks or perhaps at the ends of the dam.

The scheme of utilizing such a barrage of drains parallel with the axis of the dam and relying on it to reduce to tail-water pressure any uplift under the dam was utilized by the speaker in 1913 in the case of the Cheat River Dam (on Cheat River in West Virginia near the Pennsylvania State line). A year or two later it was similarly utilized in the reconstruction of the Stony River Dam.* More recently, it is believed, the scheme was used in the design of the Wilson Dam at Muscle Shoals.

The now commonly accepted use within the body of a dam of vertical drains and horizontal galleries intercepting the drains is referred to by the author.† Apparently he would consider them much as safeguards or "detectives" (as to deficiencies in the membrane) rather than as assurances that hydrostatic pressures within the dam will definitely be relieved along the line of such drains. The speaker is inclined to the view that there is more danger of clogging of drains within the body of a concrete dam, by deposit of material leached out of the concrete, than there is of clogging of deep drains in the foundation bed. On the other hand, the clogging of the drains in the body of a structure is believed to be of materially less consequence as concerns stability.

As to the economy which might result from the use of a water-proof membrane, such as that of which the author is a proponent, it should be borne in mind that the cross-section and hence the quantities involved in a masonry dam are frequently, if indeed not generally, determined by other considerations. Reference has already been made to the importance of stability against sliding. In addition, there is the necessity for a suitable "fattening" of the cross-section when it has to serve as an ogee-form spillway. Because of such considerations and of the means available for minimizing uplift pressures, especially under the base, by drainage, a water-proof membrane can hardly be of important effect in bringing about economy.

Nevertheless, there is a good reason for giving very serious consideration to the use of such a membrane. In case it were to insure the permanence of the concrete—if only the interior concrete—of a dam built mainly or entirely of concrete, the use of such a membrane would have ample justification. The desideratum would be a means for preventing head-water from percolating or leaking through the interior of a solid masonry dam. Granting a water-tight membrane, one might well afford to pay for the additional up-stream concrete necessary to keep the membrane or diaphragm intact even if such concrete were not relied on in determinations of stability. Evidently, such a membrane would not prevent the weathering or disintegration of the down-stream face of a dam, but it would have sufficient to its credit if it were effective in preventing head-water from penetrating construction joints and the interior concrete of a dam.

* *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), pp. 907–1100.

† *Proceedings*, Am. Soc. C. E., October, 1926, Papers and Discussions, p. 1575.

Unfortunately, the art appears as yet to offer no well-established case of a membrane that has accomplished such a purpose and has served that purpose for a substantial length of time. A membrane is subject to attack by chemical action and to mechanical difficulties resulting from differences in expansion coefficients. Considerations such as these, even if perhaps not conclusive as to the feasibility of the membrane, leave questionable any design that for stability relies on the permanence of such a layer.

The profession would benefit greatly if there were constructed and demonstrated a membrane for masonry dams that would be water-tight and permanent. Until such demonstration is available, however, the speaker would be inclined to justify the use of "water-proof membranes", if at all, by reason of their probable contribution toward longer life of the masonry rather than by reason of any economies assumed to result from reductions in design stresses and cross-sections on account of reliance on such membranes.

EXPERIMENTAL DEFORMATION OF A CYLINDRICAL ARCHED DAM

Discussion*

By Charles W. Comstock, M. Am. Soc. C. E.

Charles W. Comstock,† M. Am. Soc. C. E. (by letter).‡—Discussion of this paper seems to be inseparable from that of the paper entitled "Arched Dams"§ by the same author.

The purpose of the present paper is to compare observed deformations of an india rubber model with corresponding values computed by the process set forth in the earlier paper. These comparisons are summarized in Table 3‖ ("for base simply supported") and Table 4¶ ("for base encastrée").

Table 5 is a copy of Table 3 with the addition of a column of differences between observed and computed values and of a column of percentage excesses of observed above computed values. Likewise Table 6 is a copy of Table 4 with similar additions.

TABLE 5.—Comparison of Observed and Computed Deformations, Base Simply Supported.

Z, in inches. (1)	Computed values of u, in centimeters. (2)	Observed values of u, in centimeters. (3)	Difference, in centimeters. (4)	Percentage excess of observed above computed values. (5)
0	0.000	−0.01	−0.010	......
1	0.427	+0.47	+0.043	+10.0
2	0.468	+0.53	+0.062	+13.3
3	0.347	+0.40	+0.053	+15.2
4	0.218	+0.24	+0.022	+10.0
5	0.104	+0.10	−0.004	− 4.0
6	0.001	−0.05	−0.051	......

The author states, "the error of any reading possibly amounted to 0.03 cm." If it be assumed that the measured deformations are in excess of the actual by this amount the percentages in Table 6 would be 38, 124, 79, 57, 63, 57, and 28, instead of 141, 176, 110, 79, 80, 71, and 40, as given in Column (5).

* Discussion on the paper by B. A. Smith, M. Am. Soc. C. E., continued from February, 1927, *Proceedings*.

† New York, N. Y.

‡ Received by the Secretary, March 3, 1927.

§ *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 2027.

‖ *Proceedings*, Am. Soc. C. E., October, 1926, Papers and Discussions, p. 1600.

¶ *Loc. cit.*, p. 1601.

Even with this favorable assumption as to errors of observation, it can hardly be considered that there is substantial agreement between computed and measured distortions. If the percentage differences were nearly constant throughout, it might properly be assumed that the modulus of elasticity of the material of the model differed from the value used in the computations. However, this is not the case, and the only alternative would seem to be that the stress distributions assumed in the earlier paper, either as to the arch, or the cantilever, or both, are not the actual distributions.

TABLE 6.—Comparison of Observed and Computed Deformations, Base Encastrée.

Z, in inches. (1)	Computed values of u, in centimeters. (2)	Observed values of u, in centimeters. (3)	Difference, in centimeters. (4)	Percentage excess of observed above computed values. (5)
0	0.000	0.00	0.000	.....
⅛	0.008	0.00	−0.008	−100
¼	0.029	0.07	+0.041	+141
⅜	0.058	0.16	+0.102	+176
½	0.095	0.20	+0.105	+110
⅝	0.134	0.24	+0.106	+ 79
¾	0.172	0.31	+0.138	+ 80
⅞	0.210	0.36	+0.150	+ 71
1	0.265	0.37	+0.105	+ 40
2	0.413	0.50	+0.087	+ 21
3	0.348	0.47	+0.122	+ 35
4	0.224	0.24	+0·016	+ 7
5	0.106	0.11	+0.004	+ 4
6	−0.001	0.00	−0.001	−100

The author's analytical method consists in equating the radial distortion of a ring to the deflection of a cantilever perpendicular to the plane of the ring. The result is a differential equation of fourth order and first degree, which is integrated in terms of exponential and trigonometric functions for a cylinder of constant thickness; and in terms of hyper-geometric series for a cylinder with a thickness that varies linearly with the height.

The method is not new, having been used by Silas H. Woodard, M. Am. Soc. C. E., in the design of the Lake Cheesman Dam.* Mr. Woodard, however, contented himself with equating deflections at five or six points of the height, making no attempt to derive a general formula for deflection at any point. The principle involved is probably sound provided the assumed stress distributions for arch and cantilever are correct.

In his previous paper the author assumed† for the arch "that the bending moment vanishes throughout and that the segment is subjected to a uniform compressive stress (arch thrust) throughout its length so that the elastic compression * * * is constant throughout." This is clearly not the case. Even for the cylindrical model used in his tests, with a thickness of 17.7% of the mean radius, the circumferential stress was 18% greater at the inside surface than at the outside, although it is true that there was no bending

* *Transactions*, Am. Soc. C. E., Vol. LIII (1904), pp. 108 *et seq.*

† *Loc. cit.*, Vol. LXXXIII (1919–20), p. 2030.

moment about a line parallel to the axis of the cylinder. Had the model comprised only a segment of the circumference, instead of a complete cylinder, the assumption of no bending moment in the arch would have been obviously incorrect and the use of deflections based on such assumption entirely unjustified.

For a vertical cantilever, the author makes use of the common theory of beams. With such proportions as existed in his model this seems entirely reasonable, but for the sections of curved dams like the Cheesman, Roosevelt, and Arrowrock, or even the Shoshone Dam, there is no basis for assuming linear distribution of stress on horizontal planes; and deflections calculated by the common theory are almost certainly seriously in error.

If, as the writer believes, the author's formulas for deflections of both arches and cantilevers are incorrect, the remainder of his arch dam theory becomes simply an interesting mathematical exercise with no relation to the facts and therefore without practical usefulness. His own tests seem to demonstrate this.

The author has begun in the middle of the problem. One necessary preliminary to a proper application of his analysis is an adequate theory of the thick elastic arch. This has not yet appeared. For a structure of such proportions as the Stevenson Creek Test Dam, the ordinary theory of curved beams is probably sufficient, but for more usual proportions a more scientific analysis is needed. If the Stevenson Creek Dam had the proportions of the author's model it would be about 17 ft. thick. It is doubtful whether any one would feel justified in assuming uniform circumferential stress and no bending moment in such a structure.

Another preliminary problem, and one even more difficult of solution, is that of stress distribution in a beam the cross-section of which is rapidly varying and of the same order of magnitude as the length of the beam. When these problems shall have been successfully solved the author's method may be useful. Until that time it does not seem to have any place in the design of arch dams.

TOWN PLANNING AND ITS RELATIONS TO THE PROFESSIONS INVOLVED

Discussion*

By Noulan Couchan, Esq.

Noulan Couchan,† Esq.—The questions raised in this excellent paper are of much interest to engineers engaged in city planning. The experiences of the Town Planning Institute of Canada in general substantiate the author's conclusions. The technical qualifications required for its members are similar to those of the Town Planning Institute of Great Britain, the object being to focus technical knowledge on the question of public welfare. Thus, the engineer, surveyor, landscape architect, or lawyer who applies for membership is supposed to be qualified in or recognized by his own profession.

In some quarters there is a feeling that the architect will supersede the engineer, or *vice versa,* but the experience of the Town Planning Institute of Canada does not support this. Any city planner, no matter what his primary profession, will recognize the great advantage of collaboration with specialists in other fields. Even if the public does not recognize the need of special training, by all means the specialist should recognize his brother. For example, no one should know better than the engineer that he is not an architect, or *vice versa.*

The City of Ottawa has a Town Planning Commission of which the speaker is Chairman and Technical Adviser. Although it is an official organization, under the laws of the Province of Ontario, and has city officials as members, its powers are purely advisory and its recommendations are subject even to disapproval by the City Council. As a result, sometimes regulations and permits are made effective by Council actions, which are clearly contrary to ideal city planning. This illustrates the necessity of giving town planning the authority of law, so that it will become the will of the community and not merely the advice of an expert.

The objects of city planning, particularly of zoning, are not to crystallize, but to stabilize. The successful plan must be made elastic. Safeguards must be taken, so that changes, especially extensive changes, cannot be made overnight. In Canada, and it is hoped, in the United States, there is good reason to think that the future holds favorable promise for the effective work of the town planning engineer.

* Discussion on the paper by John Nolen, M. Am. Soc. C. E., continued from January, 1927, *Proceedings*.

† Cons. Engr., Ottawa, Ont., Canada.

THE DEVELOPMENT OF MARIEMONT, OHIO

Discussion*

By Charles Wellford Leavitt, M. Am. Soc. C. E.

Charles Wellford Leavitt,† M. Am. Soc. C. E. (by letter).‡—Inasmuch as Mariemont has been typified in Mr. Fay's paper as a "practical example of the application of the principles of community development" and as being "unique in the history of town planning in the United States" and is called by the author, "A National Exemplar", it would seem as if a little more should be known in detail about certain statements in the paper which are of great interest to town planners, particularly if Mariemont is to be admitted as the standard to follow in planning residential communities.

Is it possible to set out a complete National example in community building? Each problem presents separate and independent conditions. Topographically, Mariemont lends itself to a maximum of principles governing city planning and the planners themselves have been so well grounded in their profession as to be able to make full use of all possibilities.

The general principles of design as expressed in the paper impress one as sound. Many if not all of them have been used elsewhere and satisfactorily, although the writer knows of no instance where they have all been used together in harmony as at Mariemont. In bringing all the most improved methods of design and execution into co-operation as has been done in this case so successfully, great credit must be given to the planners who have thus made a distinct advance in town planning.

It might be of great interest to enlarge upon the statement that "it does not provide homes for the very poor; experience thus far has shown the practical impossibility of providing new homes for people of the lowest economic scale". It is also stated that "Mariemont is intended primarily as a place of residence for families of widely different economic standing, and especially wage earners". Is it to be understood from these statements that Mariemont only accommodates the well-to-do and those of moderate means? Some one must perform duties which return small pay (though a living wage) and of these there are many, and they are necessary for the functioning of any community, whether residential or industrial. Where are these people to live if not in Mariemont? To some extent these people, not very poor, but of less than moderate means, have been provided with new houses elsewhere

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* This discussion (of the paper by Frederic H. Fay, M. Am. Soc. C. E., presented at the meeting of the City Planning Division, New York, N. Y., January 21, 1926, and published in October, 1926, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Civ. and Landscape Engr. (Charles Wellford Leavitt & Son), New York, N. Y.

‡ Received by the Secretary, January 13, 1927.

in other new towns. Could it not have been done in Mariemont? Or has it been done? Almshouses are the only examples known to the writer of new structures having been provided for the very poor, those of the lowest economic scale. With this exception the very poor have always been obliged to house themselves, usually in the houses discarded by the well-to-do, who build and live in new homes.

It would be interesting to be informed of the rentals received for the smaller houses, or those "many small flats or apartments, some even a single room", provided to "meet the needs of those who can afford only low rentals". What is the amount of these low rentals?

The church, the recreation facilities, the housing so carefully planned by various architects are as nearly perfect as modern art can devise them and should give the greatest comfort and satisfaction to the inhabitants of Mariemont.

The street system seems perfect except possibly the direct "major thoroughfare from Cincinnati passing through the town center". It would be interesting to know if this major thoroughfare might not have passed outside the community with a branch into the town center so as to keep out the through traveler, rather than congest the town center with people having no business in Mariemont.

The town plan; the engineering, with the distribution systems for the water, gas, electricity, telegraph, telephone, and police and fire alarm; the collection systems of drains and sewers; and the street pavements seem to be beyond criticism. The installation of service pipes to every lot before the street pavements are laid is especially to be commended.

The central heating plant, the one service which is developed in Mariemont from the beginning to the delivery of steam into the public buildings and homes, is certainly the distinctive engineering feature. Can this be run with such economy as to compete with ordinary methods? If so (and the cost figures on this would be interesting), it will be a godsend to the coal heaver and the ash remover.

While it is stated that Mariemont is not a philanthropy, surely the munificent act of Mrs. Emery in providing the community buildings, such as the church, school, hospital, parks, playgrounds, streets, and other features of a public nature, as "her gift to the new town", relieves the residents of the necessity of financing these facilities which are a necessity in any well-ordered community.

The things that have made for progress, those things that are appreciated best, and from which the greatest amount of good, pleasure, and enjoyment are derived, are those provided by one's own efforts and funds. With this great gift and the fact that the profits from this development "are restricted to a low figure", will not this community be spoiled by not paying for those facilities which other communities do provide, and in which they take pride because they have done so? With so much provided for them will not Mariemont's residents have little left to do but to sit back in luxurious ease and eventually become stagnant and decadent?

It will be of great interest to all town planners to know whether Mariemont develops a soul; and yet how else can such perfection be secured in the development of a residential community which so frequently happens rather than is planned?

Fully realizing the noble motive that prompted Mrs. Emery in founding Mariemont and in making these magnificent gifts for real and lasting service for the better housing of mankind, the writer insists that this is not "a real estate development on normal American lines", but that an atmosphere of charity will always pervade Mariemont.

In order that Mariemont, or any other place which has been brought into existence by the act of an individual, shall have a "soul" and the spirit of co-operation and advancement—assuming that all things are done as Mrs. Emery has performed in Mariemont, so carefully, thoroughly, and generously—it might be necessary to insist that the residents in this town pay for what they get, at market prices, so that patronism does not become evident, and the characters of the individuals will be developed. We must not have indolence, but virile active workers if communities are to keep pace with the general world advancement. The soul of the town and the spirit of freedom and the buoyancy of advancement will be measured by the strength of the people of the community and their activities.

Mariemont may serve a purpose to determine how far modern art and science which here have been so skillfully combined may go and yet develop a spirit to carry on the traditions which have made the towns and cities of the United States what they are.

It would seem that if Mariemont's citizens were to carry the whole financial burden and if accommodations were provided for those of less than moderate means, this very beautiful conception will truly become an "examplar" for all to follow.

THE CINCINNATI CITY PLAN IS NOW LAW

Discussion*

BY THOMAS H. REED, ESQ.

THOMAS H. REED,† ESQ. (by letter).‡—It would be ungracious, indeed, to seem to detract from the felicitations due to Cincinnati on her enviable acquisition of a legally established city plan. So much planning effort has got no further than glossy reports and elaborate diagrams, that a plan in force is a marvel to be heartily admired. Mr. Ford himself, however, has deliberately raised the general question of the wisdom of permitting planning commissions to make plans which have in effect the force of law. To this question the writer would return an emphatic negative.

One can readily pardon the city planning expert for exalting his specialty to a place of equality with the other aspects of city government taken together. Experts in education, public health, etc., are prone to do the same. The fact remains, however, that city planning is only one of many functions of the modern city. That a wise plan is essential to the successful performance of many of these other functions does not remove it from this position. It is no more essential to happy city life than many other services, such as those of the departments of health, fire, and police. There is nothing in the nature of the planning function itself to distinguish it from many other city activities. City planning is a technical art calling loudly for the leadership of experts but so are the construction of sewage disposal works, the development of transportation systems, or the conduct of diagnostic laboratories. Should, then, there be established for city planning a procedure at variance with that used for other municipal purposes?

An affirmative answer to this query obviously requires support by reasons of extraordinary cogency. These reasons must have relation not only to the problem of city planning, but to the infinitely more difficult and obviously more important problem of city government as a whole. It is entirely conceivable that a method which will give most quickly a well-established city plan, may not in the long run, prove consistent with the best management of the affairs of the city. Indeed, municipal reformers, during the last half century, have waged a furious battle against special jurisdictions—park boards, water boards, sewer boards, and what not—each of which was justified (at least, originally) by the necessities of a particular service, but all of which taken together have tended to destroy the responsibility of the general government

* Discussion on the paper by George B. Ford, Esq., continued from February, 1927, *Proceedings*.

† Prof. of Political Science, Univ. of Michigan, Ann Arbor, Mich.

‡ Received by the Secretary, December 8, 1926.

of the city. If there is a conflict of interest between planning and sound principles of municipal organization any doubts should be resolved in favor of the latter.

In all systems of municipal government except American, the council has been the real governing body of the city and respected as such. In the United States, early in the Nineteenth Century city councils began a descent which resulted ultimately in their being deprived of most of their earlier powers and in the almost total loss of responsibility and public respect. No better example of this situation can be found than the position of the Cincinnati Council prior to January 1, 1926. It consisted of thirty-one members most of whom were entirely incapable of forming an independent judgment on the government and affairs of the city. In fact, the Council had almost totally abrogated the function of determining city policies, which might normally be supposed to belong to it, in favor of the Hamilton County Republican Committee. It was not until this tolerably enlightened but wholly extra-legal committee had placed the sign of its approval on it, that substantial progress could be made toward the establishment of a city plan. This Republican organization controlled absolutely not only the Council, but the Mayor and all his appointees.

It is not possible to do without a municipal legislative body. Somewhere the policies of the city must be determined, if not in a popularly elected and responsible body, then somewhere else. Very little support could be found for the proposition to abolish the city council. There is, however, but one alternative, that is, to put it on a respectable basis. It is not necessary to examine in detail the methods available for this purpose. It is enough to point out that the council cannot be made respectable by withholding from it legitimate legislative functions. In city government, administration of necessity occupies a more prominent place than legislation. The city is engaged primarily in rendering a multitude of services which involve few questions of general policy and innumerable questions of administrative application. Under these circumstances it is no wonder that the city executive, be he mayor or manager, bulks larger in the public eye than the council. There is all the more reason for leaving with the council as fully as possible the determination of such matters of general policy as belong to city government.

City planning is obviously a legislative function. It involves the laying down of a permanent policy of city growth. Mr. Ford and other city planners will agree that it is the most important subject of municipal legislation. To remove it almost altogether from the competence of the council is to strike a deadly blow at the vitality of that body. It may well be admitted that city planning is a function for the performance of which a council needs technical advice. Advice, however, is essential to good legislation of almost every conceivable kind whether enacted by the Congress of the United States or by the council of the smallest city. The conditions of modern life have become so complicated that no body of men is capable of deciding well, on the basis of its own unaided discretion, any considerable number of the questions to which government gives rise. Until it is safe to leave each subject of legislation to a soviet

of the experts in that subject, however, it is necessary to have legislatures. By no other means yet discovered can the interests of the community as a whole be represented in legislation.

The writer admits that the old Council of Cincinnati was entirely incompetent to deal with the plan of that great city. In another place* he has indicated his doubts of its ability to perform any representative function. This is by no means true of the Council of nine which took office in January, 1926. This Council was elected at large and is adequately compensated. As a result of the civic stimulus which often accompanies a new charter it is probably as able a council as ever directed the affairs of a large city in this country. In the choice and control of the city executive (a manager) it exercises powers which make service on it worth while to men of broad outlook. The way to good councils, in other words, is by increasing, not decreasing their importance.

Mr. Ford suggests an analogy between the adoption of a city plan and the regulations of a board of health, of a police board, or of a fire commission. The analogy is very weak. None of these bodies has anything but a very subordinte power of making regulations which must in every case conform to the State laws or city ordinances relating to the subject. These laws and ordinances are more often than not very detailed in character, leaving only a modicum of discretion to the board. A city plan is not only a more fundamental matter than the angle at which automobiles must park along the curb line; it is a subject with which State Legislatures have dealt only in the most general terms. If the council cannot legislate with regard to it, the discretion of a planning board, empowered as is that of Cincinnati, would be well-nigh absolute.

The Standard State Zoning Enabling Act, issued by the U. S. Department of Commerce, provides in its first section for vesting the power of zoning in the "legislative bodies of cities and incorporated villages", and it is rumored that the proposed Standard City Planning Enabling Act will be similarly deferential to the normal prerogatives of the city council. The attitude taken by the Advisory Committee on City Planning and Zoning, appointed by Secretary Hoover, which is reflected in these acts, seems to represent a sounder view than that of the Ohio Planning Law.

* "The Government of Cincinnati and Hamilton County," 1924, pp. 189, *et seq.*

THE NEW YORK STATE BARGE CANAL AND ITS OPERATION

Discussion*

By Frank L. Bolton, Assoc. M. Am. Soc. C. E.

Frank L. Bolton,† Assoc. M. Am. Soc.. C. E. (by letter).‡—For several years the writer has been hoping that the problems confronting the New York State Barge Canal would be opened for discussion before the Society. The true facts concerning the Canal have been most difficult to ascertain as a result of so much erroneous information being in circulation. All those who have the interest of the Canal at heart, will welcome this paper, coming from one who knows whereof he speaks.

Furthermore, at this time, when there is an awakening interest in internal waterways, the paper is particularly opportune. The author's history of the New York State canals and his description of the physical features of the Barge Canal, enables one to visualize the growth over 110 years, from a "ditch", 4 ft. deep and 28 ft. wide, to the present magnificent waterway, about 800 miles long (with connecting rivers and lakes) and costing the State of New York $175 000 000.

The writer hardly realized that 70% of the population of the State of New York lives within about 2 miles of the Canal System. This fact puts the canals in an economic position of great importance and indicates that, when the waterway is functioning properly, the benefits therefrom will be widespread and effective upon a very large proportion of the people of the State.

The author has pointed out that there are not enough boats on the Canal at the present time and concludes that "this lack of carriers is the main reason why the Barge Canal is not transporting the tonnage for which it was designed and to-day is capable of carrying", giving several reasons.§

In this viewpoint the writer fully concurs. There are, however, other factors of more importance that have discouraged boat operation. They are so fundamental that, if properly recognized, they will answer the question of "What is wrong with the Canal?"

Before proceeding to answer this question, consider what can be said in favor of the Canal, as it is to-day.

* Discussion on the paper by Roy G. Finch, M. Am. Soc. C. E., continued from January, 1927, *Proceedings*.

† Ithaca, N. Y.

‡ Received by the Secretary, January 4, 1927.

§ *Proceedings*, Am. Soc. C. E., October, 1926, Papers and Discussions, p. 1682.

The writer is offering this discussion from a dual viewpoint, first as an engineer interested in water transportation in general and in the physical features of the Barge Canal in particular; and, second, as a shipper of freight, who is now using the Canal in a satisfactory way as a dependable, economical means of transportation.

From a physical standpoint, the Canal appears to be a practical working waterway. Without doubt there are some changes and improvements necessary but these minor defects can be corrected without any large expenditure of money by the State. As an illustration, the condition at the outlet of Cayuga Lake may be mentioned. At this point the lock walls (Mud Lock) are so constructed that when Cayuga Lake is at its maximum stage, the water is about a foot higher than the top of the lock walls and it is impossible to lock boats through. This defect in design can be remedied by raising the lock walls and the lower lock-gate 1 ft. or 2 ft. During the past few seasons of navigation, there have been delays at this point because the boats could not be locked through until the lake was drawn down or until a temporary dam had been constructed along the top of the lock walls.

As to the depth of water in the Canal, about which so much erroneous information has appeared, the writer has, during the past two seasons of navigation, loaded boats to a 10.5-ft. draft. These boats have moved without difficulty from a point on Cayuga Lake six miles north of Ithaca, to New York City, as well as westward to Tonawanda and Niagara Falls.

Having shipped 100 large barges loaded with salt, averaging from 500 to 700 tons in capacity, and having found that the movements of these boats proceeded in a dependable manner, running on a satisfactory schedule, the writer cannot see that any pertinent criticism of the physical conditions of the canal can be raised. It is evident, therefore, that an answer to the question as to what is wrong must be found elsewhere.

Obviously, capital must be interested in canal operation, if a sufficient number of boats are to be forthcoming for handling the available tonnage. The problem of interesting capital to-day is far different from the situation in the old days of operation on the Erie Canal. When the boats operating on the Canal had a carrying capacity of only 200 or 300 tons, or less, the motive power was horse or mule teams. An individual with small capital could finance the construction of one or more small boats and with a team of mules could tow them along the Canal and make a living. The boat owner usually made his home on his boat both summer and winter and with small expense of operation the canal thus offered an easy line of independent endeavor for many individuals. Under these conditions, many "Canalers" entered the trade with their own small equipment and as a result ample boats and much competition was available for the Canal.

To-day, with the enlarged Canal, the tow-path is gone and with it most of the small boats and the individual operators. Those desiring to enter into canal transportation must obtain large amounts of capital in order to secure larger and more modern boats, expensive towing equipment, etc. As the author has pointed out there are single-motor ships operating on the Canal

to-day, that cost $175 000 per unit. Here is the difficulty in the present situation, capital has not been interested in a large way in going into Canal transportation. If it can be shown how capital can be interested, and that the undertaking will be profitable, it is believed that the problem of the Canal will be solved.

In addition to the reasons pointed out by the author, as to why capital has not been interested, the writer wishes to add another of vital importance. This is the matter of a definite continuing policy for the Canal, as well as a permanent personnel for its operation and maintenance. The writer hopes that the author, in his closing discussion, will enlarge on the question of a continuing policy.

The canals are operated by, and under, the administration of the Department of Public Works of the State of New York. This Department has many other duties besides looking after the Canal. The State Highway Department, the Department of Public Buildings, and the duties of the State Engineer and Surveyor all come under it. Inasmuch as a large portion of this Department's work is highway work, and as many of its engineers are highway engineers, it is logical that the highway point of view predominates in the operation of the Canal. The attitude of the State, as expressed through the Department of Public Works, seems to be this: "Here is the Canal ready to be used, please come and make use of it". Obviously, this attitude will not sell the Canal to those people who can use it, as they want to know whether the conditions now obtaining as to physical features as well as the favorable attitude of the present operating personnel will persist over a long enough period of years to justify their operations on the waterway. No criticism is intended on the present operating personnel; the writer's point of view is that the Canal does not have a permanent personnel engaged solely in advancing the interests of the waterway.

As now organized, those in authority, having to do with the policy and operation of the Canal, change with every change in the State Administration, or more often, and this militates against a continuing policy. The difficulty of those using the Canal in having to deal with a constantly changing personnel may be illustrated by referring to the defect in the design of the lock walls at the outlet of Cayuga Lake mentioned previously. This defect was called to the attention of the State Engineer several years ago, but before steps were taken to remedy it, he passed out of office. The matter was again called to the attention of the newly elected State Engineer some months ago, and as his term of office has expired, it will probably be necessary to take the matter up again with another official, in order to get a minor engineering defect corrected. With a continuing personnel, shippers would be doing business all the time with the same individuals and would have a much better chance of getting results.

A continuing policy, as well as a continuing personnel, must be provided, if this splendid waterway is to develop properly the tonnage for which it was designed and which appears to be available. It would seem that the State of New York, having spent $175 000 000 for this Canal System, would

see fit to place this large investment in the hands of a permanent board or authority, to be operated on a business basis and in much the same way that a railroad would be operated had it a capital investment of $175 000 000.

The Governor of the State of New York is now advocating a quasi-public organization, to be known as a "Water Power Authority" for handling the water-power situation in the State. Why not a "Canal Authority", organized along similar lines, for administering this investment of $175 000 000?

Various suggestions have been made from time to time by those interested in some phase of canal transportation, looking toward further expenditures by the State for docks, terminals, tipples, freight-packet service, and similar items, with the thought that these things are needed for making the Canal more useful. The writer is opposed to any such expenditures. These matters will be taken care of and should be taken care of by private capital. If assurance can be given that the physical conditions of the Canal will be maintained over a period of years and that a continuing policy and organization will be provided, it appears inevitable that capital will become interested in the Canal, as positive economic advantages to shippers of freight and to operating companies can be shown by past experience.

The writer cannot agree that enlarging the present Canal would prove a greater inducement to capital to place boats in operation. The suggestion was made that deepening the Canal from the present 12 ft. to 15 ft. and increasing the width of the channel to 110 ft., would allow self-propelled barges of more than 2 000 tons capacity to operate efficiently and the carrying capacity of the larger barges to be increased 20 to 30% with practically no increase in operating expenses. With the possible exception of grain traffic, the writer can see no economic advantage in being able to operate single units of 2 000 tons capacity. For general use on the Canal a unit of 2 000 tons of cargo is larger than can economically be handled by the shippers and receivers of freight. A fleet of four barges of 500 tons capacity each is a much more flexible arrangement for practically all commodities, except possibly grain. Looking to the future, it is the writer's belief that as the tonnage on the Canal develops, the grain movement will constitute a lesser percentage of the total than at present. One of the problems facing the shippers of many commodities is the inability of the consignee to take in and handle at one time as much as 500 to 700 tons of freight. Furthermore, the smaller units now in use on the Canal enable the shipper to dispatch a fleet of several units and distribute them along the Canal at different points without tying up the motive power during the time required for discharging cargo at local points.

It is true that the cost of operating on the Canal is reduced rapidly as the tonnage handled at one time increases, but it is not necessary for low costs that the tonnage handled at one time be contained in one bottom. The author cites one carrier, that operates a fleet of five barges and a steamer utilizing the full capacity of the locks on a draft of 10 ft., 4 in., with a carrying capacity of 3 600 tons. With such an operation now possible, the writer cannot see

where any greater inducement to capital would be offered by enlarging the Canal to such a point that 2 000 tons capacity could be handled in one bottom.

The present Barge Canal can and will justify itself as a dependable waterway independent of, and without consideration for, any deep waterway that may be constructed from the Great Lakes to the Atlantic. It will of necessity be many years before the proposed deep waterway can be put into operation even if construction should be initiated within the next few years; and after the completion of such a deep waterway, the present Barge Canal, if handled properly, will become an important feeder for the ship canal as well as serving locally the territory along its course. The question as to what will happen to certain tonnage on the Barge Canal at the end of ten to fifteen years, when a ship canal might be in operation, has little to do with arriving at a proper solution of the Barge Canal problems at this time.

Recognition should be made of the fact that a considerable tonnage is now being handled over the Barge Canal by shippers along its route, and that these shippers have made investments in the way of docks, storage warehouses, and other items of plant and equipment necessary for shipping and receiving freight over the Canal. These investments have been made in good faith with the belief that the State would get its waterways in such shape as to be continually useful. These shippers must be considered pioneers, who are pointing the way to a broader and more general use of the State's waterways and, as such, are entitled to a definite and fixed policy on the part of the State concerning the present and future status of the operation of its waterways.

QUANTITIES OF MATERIALS AND COSTS PER SQUARE FOOT OF FLOOR FOR HIGHWAY AND ELECTRIC-RAILWAY LONG-SPAN SUSPENSION BRIDGES

Discussion*

By Messrs. Allston Dana, I. Oesterblom, Francis P. Witmer, Leon S. Moisseiff, V. A. Eberly, Horatio P. Van Cleve, and Wendell P. Brown

Allston Dana,† M. Am. Soc. C. E.—The method used by the author for determining the weight of stiffening trusses has already been discussed. As the speaker understands it, the trusses were designed for convenience by the so-called elastic theory (which has been known for some time to give unnecessarily heavy trusses), and the weights were then scaled down by certain ratios so as to obtain weights corresponding with trusses designed by the so-called deflection theory (which has been recognized as giving correct results). The ratio used by the author was that found in the case of the stiffening trusses used in the Delaware River Bridge. These trusses were designed by the deflection theory, and the comparison with trusses designed by the erroneous elastic theory was made later only as a matter of interest and to bring out the "saving" resulting from using a correct analysis.

This correct analysis is not really based on any particular theory but rather on the actual conditions which must obtain. These conditions are that under the influence of temperature and live load, full or partial, the cables change shape or deflect, and at all points the trusses must deflect the same amount. The distribution of the total load between the cables at each point must be such that their deflections are the same throughout. This distribution may be found by a cut-and-try process or by more formal methods. If there were no trusses the cables would deflect so as to conform to the force polygon of the loads. This natural deflection is dampened or resisted by the use of stiffening trusses to a greater or less extent, depending on the relative stiffening effect of the trusses.

Two extreme cases may be considered. First is that in which the trusses are relatively so stiff as compared with the cables that practically they resist entirely the deflection of the cables. This is likely to be the case for very short spans, where the large ratio of live load to dead load would make very stiff trusses desirable. For this case the elastic theory would be practically correct, because this theory assumes that there is no deflection of the cables or, in other words, that partial live loads are uniformly distributed by the trusses to the cables.

* Discussion on the paper by J. A. L. Waddell, M. Am. Soc. C. E., continued from February, 1927, *Proceedings*.

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The second extreme case is that in which the trusses are relatively so flexible that they offer practically no resistance to the deflection of the cables. This is likely to be the case for very long spans where the small ratio of live load to dead load would make stiff trusses unnecessary. For this case the design of the trusses would consist merely in finding the unit stresses in the chords corresponding to the curvature into which the trusses would be forced by the deflection of the cables. The unit stress for any given amount of curvature is directly proportional to the truss depth, and obviously a depth would have to be selected which would give permissible unit stresses. The determination of the chord areas in such a case would be a matter of selecting a nominal cross-section, of sufficient size to justify the compressive unit stress.

These two extreme cases consist then either in assuming absolutely stiff trusses and designing them for given loads, using a depth to give an economical design, as in simple truss bridges; or in assuming absolutely flexible trusses and designing nominal sections, with a truss depth and unit stresses that correspond. Of course, no trusses are either absolutely stiff or absolutely flexible, but for very short spans the first assumption would be approximately correct, and for very long spans the second assumption would be approximately correct. In this latter case, the weight of trusses would be many times lighter than those assumed by the author. For what may be called intermediate lengths, however, neither of these methods would be satisfactory. The Delaware River Bridge is a good example of an intermediate case because the trusses as designed reduce the cable deflections to about one-half the natural or unstiffened amounts. To have designed the trusses by the first method would have given a result not even approximately correct. To have designed them by the second method would have necessitated a depth of truss about one-half that actually used, in order to keep the unit stresses down to the permissible amounts. Such shallow nominal trusses were not considered adequate, however, as it was felt desirable to resist to a considerable extent the motion of the cables and towers. There remained then no escape from the somewhat more complicated method of correctly analyzing the deflections of the combined cable and truss system, and finding the load distribution and the bending moments in the trusses. This may be looked at as a process of selecting both the truss depth and the allowable unit stresses, and then of computing the necessary chord areas to give trusses of such stiffness that the cable deflections will be dampened sufficiently to keep the resulting truss curvature down to that corresponding with the depth and allowable unit stresses. If somewhat shallower trusses had been used their weight would have been much less, or if somewhat deeper trusses had been used their weight would have been much greater, so that the particular ratio found between the weight of those used and that of trusses designed by the elastic theory is a function of the depth.

For the author to take the "ratio" found for the Delaware River Bridge for the purpose of finding truss weights for all lengths of spans is, therefore, equivalent, not only to agreeing with the depth used for that length of span,

but of tacitly assuming certain depths for all other span lengths, depths which there is no reason to believe are the same as those previously assumed in Table 1* of the paper. The very pertinent criticism that the truss weights as given in Fig. 5† are much too great for the longer spans is therefore equivalent to stating that the truss depths as given in Table 1, or as indirectly established by the assumed "ratio", are unnecessarily large for the longer spans. With this the speaker is in agreement, it being his opinion that a truss depth of about 30 ft. would give a very satisfactory truss for all length of spans.

With regard to the use of over-head bracing the author states† that in all the designs "he has used over-head bracing, as, on general principles, he is opposed to pony trusses, especially in long spans". He further states:

"It is true that the tension in the suspenders tends to keep the top chords of pony trusses from getting greatly out of line; but when computing the top chords, it would be difficult to determine satisfactorily the proper value of the slenderness ratio".

In suspension bridges with only two trusses the trusses are naturally quite far apart and over-head bracing if used would be very heavy and would also add to the cost of the cables and towers. It seems unfortunate, therefore, that the author has dismissed so summarily the matter of omission of over-head bracing and for the reasons given.

The top chords of pony trusses are held in line not by the tension of the suspenders but by the web members of the trusses, especially the verticals; and the adequacy of such support depends on the depth of the trusses and not on the length of the span. The length of span hardly enters into the question, for, as mentioned previously, the truss depth is or should be independent of the span length. In the case of the Delaware River Bridge it was decided to omit the over-head bracing both for economy and for appearances. The speaker made a thorough investigation of the stresses in the top chord and verticals due to the omission of the bracing. The verticals not only hold the top chord in line but also tend to force the chord out of line due to the deflection of the floor-beams from live load. With a floor-beam fully loaded between trusses and the cantilever projections empty, the maximum inward deflection of the verticals occurs. With the cantilevers fully loaded and with no load between trusses, the maximum outward deflection of the verticals occurs. Assuming alternate loading condition of this kind along the bridge, the top chord would be forced into a series of lateral waves. The shortest distance between points of contraflexure is two panels. For this case it was found that the lateral stiffness of the chord was sufficient to resist almost completely the deflection of the verticals, forcing them to bend back. The lateral force caused by the change in direction of the chord under compression was of course included in the calculations.

Where the points of contraflexure were far apart, however, the chord was found to be insufficiently stiff to resist deflections and the lateral thrust from the change in direction of the chord deflected the verticals somewhat more,

* *Proceedings*, Am. Soc. C. E., November, 1926, Papers and Discussions, p. 1768.
† *Loc. cit.*, p. 1770.

being resisted by the stiffness of the verticals and floor-beams. Cases were investigated for various distances between points of contraflexure and the bending stresses in chords and verticals computed. In no case was the chord bending stress found to be greater than 3 000 lb. per sq. in., and the allowable compressive unit stress was reduced by that amount for designing the chord members. The value of the slenderness ratio mentioned by the author as being uncertain for pony trusses, thus did not have to be determined.

I. OESTERBLOM,* M. AM. SOC. C. E. (by letter).†—The writer recently had occasion to inquire into the matter of light-weight concrete, and his attention was called to "Haydite", to which Mr. Waddell makes reference,‡ and regarding which significant claims are made, evidently well substantiated by both experience and tests. At the time of the inquiry copies of several test certificates were given, a few extracts from which may now be published.

TABLE 3.—TESTS ON "HAYDITE" CONCRETE.

Reference.*	Test No.	Mixture.	Wetness, percentage.	Weight, in pounds per cubic foot.	Compressive stress, in pounds per square inch.	Tensile stress, in pounds per square inch.	Age of sample, in days.	Notes.
A..	H H 253	1:3 :5	3.5	93	1 290	....	28	....
B..	H H 253	1:3½:5	3.5	101	1 621	....	28	....
C..	30 846	1:2 :2	21.6	1 048	3 620	....	28	....
D..	30 846	1:1 :2	20.4	106	4 350	....	28	....
E..	....	1:1¼:1¼	....	112	3 360	....	....	E = 1 940 000 lb. per sq. in.
F..	1920-37	1:3	....	....	....	418	31	....
G..	1920-37	1:3	....	....	....	567	90	....
H..	1920-27	1:2 :2	....	....	3 308	....	28	....
I...	1920-27	1:2 :2	....	....	3 919	....	90	....
J...	1920-26	1:2 :4	....	....	2 065	....	28	....
K..	78 409-79 061	1:2 :4	"Normal"	107	3 268	....	30	....
L...	82 028	1:2 :4	"Plastic"	115	3 082	....	30	....
M..	82 309	1:1 .7:2.	"Plastic"	118	4 523	....	30	....
N..	87 831	1:2 :4	"Field"	116	2 520	....	28	....
O..	88 571	1:2½:3	....	97	2 738	....	28	110½ lb. per cu. ft., wet.

* *A* and *B* = Kansas City Testing Laboratories, Inc.; *C* and *D* = Bureau of Standards, Washington, D. C.; *E* = Lewis Inst., Chicago, Ill.; *F* to *J* = Municipal Laboratory, Kansas City, Mo.; *K* to *O* = Kansas City Testing Laboratories, Inc. On control tests for actual building operations as follows (all in Kansas City); *K* = Deaner Dental Clinic Bldg.; *L* = Westinghouse Bldg.; *M* = Kansas City Star Bldg.; *N* = Seested Bldg.; *O* = Central Junior High School. The test reports often had the remark that the samples were weighed wet.

Table 3, therefore, is presented containing extracted averages in regard to weight and strength, also information about mixture, consistency, and the ages of the test pieces. Reference is also made to the nature of the tests, many of them having been made as control tests on actual building operations; the names of the laboratories are also given.

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† Received by the Secretary, January 31, 1927.

‡ *Proceedings*, Am. Soc. C. E., November, 1926, Papers and Discussions, p. 1765.

From Table 3 it will be seen that the compressive strength is considerable. Evidently, the "Haydite" aggregate itself is equal or superior to the enclosing cement mortar. The same is true, perhaps in a larger measure, also in regard to the tensile strength. The modulus of elasticity is quite equal to normal.

There are additional qualities, shown in the certificates, but not noted in Table 3, which are also noteworthy. "Haydite" concrete is most thoroughly water-proof, which ordinary concrete even when well made is not; it is also a much safer material in very intense fire.

Most of the commonly used aggregates have been produced by the colloidal process; they are, therefore, full of capillaries and very water-absorbent. The cement mortar is similar in nature and likewise water-absorbent. The "Haydite" aggregate is produced by a burning process; it is full of bubble-like cavities, or small cells, but these are enclosed by walls, produced in fire and perfectly impervious. It is in the nature of a cellular material to aid in a more thorough hydration of the cement, thereby densifying the structure of the mortar. The two features combine to make "Haydite" concrete very impervious.

It is of less significance in bridge construction that "Haydite" concrete is especially resistant to intense fire. It is obvious that it should be so. Having been produced in intense fire there can be no further changes of a chemical nature when the material is again exposed to intense heat. "Haydite" is, therefore, proof to both elements, fire and water.

If the future should bear out what the past already seems to have well established, it is clear that "Haydite" concrete is the material for all bridge floors, and indeed for supporting elements as well in girder and arch bridges. The more the dead weight enters as an important element of construction, the more necessary will be the use of "Haydite" concrete or some similar material possessing equal advantages.

FRANCIS P. WITMER,* M. AM. SOC. C. E. (by letter).†—The writer is particularly impressed with the author's conclusions as influenced by the very startling claim of saving in weight of stiffening trusses, through the use of the "deflection theory" instead of the "elastic theory" in their design. Such a saving as 64% in weight of trusses in the case of a large bridge seems hardly credible, and the writer is led to question the correctness of this percentage.

The explanation of the deflection theory given by Mr. Moisseiff is a necessary supplement to the paper itself. Referring to his example, it was asserted that "multiplying the reduction in H (784 000 lb.) by 255 ft. gives a relieving moment of 200 000 000 ft-lb. which will reduce the bending in the truss". Here the writer fails to follow the argument; he cannot see the justification of thus multiplying the difference between the values of the horizontal dead load pull in the cable for two different versines by either of these versines in order to obtain the value of such a relieving moment. The

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* Director, Civ. Eng., Univ. of Pennsylvania, Philadelphia, Pa.
† Received by the Secretary, March 4, 1927.

total moment at the center of the stiffening truss from the dead load cable stress before the live load was applied would be,

$$40\,000\,000 \times 250 = 10\,000\,000\,000 \text{ ft-lb.}$$

and the moment from the dead load cable stress after the live load was applied would be,

$$39\,216\,000 \times 255 = 10\,000\,000\,000 \text{ ft-lb.}$$

As these moments are equal, the relieving moment from dead load really appears to vanish entirely.

The writer has made no quantitative comparison between the actual truss stresses in any given case under the two methods of calculation, and is in no position to question results which others may have obtained by a complete application of each method throughout a specific design. If, however, the percentage of saving alleged in the paper has resulted from a consideration of a dead load relieving moment obtained as described, he seriously doubts whether any such percentage of saving or, in fact, any saving of appreciable proportions will result by the use of the "deflection theory" instead of the "elastic theory."

LEON S. MOISSEIFF,* M. AM. SOC. C. E.—This paper on the cost of long-span suspension bridges is of much interest to bridge engineers. The author has heretofore been the chief contributor to the studies on bridge economics published in the United States. This paper thus forms the logical continuation of his investigations.

Studies in bridge economics in English are rather scant and of such as have been made only a few have been published. Engineers of other countries have given much attention to the subject but their studies have been practically limited to short-span bridges of the simpler types. For this there was good reason. When Governments own the railroads and control by decrees the applied loads and the allowable materials and stresses and prescribe the standards of the designs to be followed, the study of the cheapest bridge within the law becomes much simplified—a matter of comparing types and spans based on exactly the same requirements and the same allowances. In fact, European studies are frequently referred to as based on the "Regulations of A. D. so and so". A change in the Governmental decrees is, therefore, soon followed by a new study of the economics it involves.

For the same reason European engineers, with a few purely academic exceptions, have not gone deeply into the study of the economics of long-span bridges. Another more important reason is that there are not enough long-span bridges on the European continent to necessitate such studies. The same reason holds true in the United States for suspension bridges of the enormous spans investigated by the author. Therefore, the necessity of this paper and its use for suspension bridges of very long spans is not obvious. Suspension bridges of the kind studied are large and costly; they are not built every day. During the last twenty years, that is, since the construction of the Manhattan Bridge, few large suspension bridges have been built

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in the world. They can be counted on the fingers of one hand: The Bear Mountain and Delaware River Bridges, in the United States, the Florianopolis Bridge, in Brazil, and the Cologne Bridge, over the Rhine.

Long-span bridges are expensive either relatively because of the light available traffic or absolutely because dense traffic means wide roadways, heavy loads, and thickly populated cities with correspondingly expensive land. Projects requiring large funds take considerable time to ripen. The demand for them must be persistently felt before public or private agencies undertake their realization. Some well-known bridge projects have taken decades in developing. Engineers, therefore, will have more than enough time to study the proposed bridge and estimate its cost. They will hardly be compelled to read off the cost of the project from diagrams based on materials, fabrication, and estimates of cost, which may be established in the year of grace 1926.

All these remarks do not in the least lessen the intrinsic value of the paper. A great amount of work has been done by the author and when the occasion arises the planning engineer will make good use of it.

In his study of stiffening trusses the author refers* to the speaker's paper on the Camden Bridge, showing that "for the stiffening trusses of the main span, 1 750 ft. in length, the deflection theory reduces the weight of the metal to 64% theoretically, or 67% actually, of that found by the elastic theory, the difference in these percentages being due to the use of certain minimum sections in designing."

This is, therefore, an opportunity to make a short statement on the deflection theory. This theory differs from the elastic theory, as developed by Mueller-Breslau, Melan, and others, in that in the latter it is tacitly assumed that the cable curve remains undistorted after the advent of load on the bridge, while the former takes cognizance of the distortion of the structure. Thus it follows that in the elastic theory the effect of the dead load in stiffening the bridge is neglected, while in the deflection theory it is taken into consideration and exerts an important influence on the results.

The main considerations of the deflection theory can be explained without recourse to mathematical symbols. The suspended trusses together with the cables, towers, and anchorages form an elastic structure acting as one system, that is, an external cause affects the members of the entire system elastically and on its passing the system is fully restored to its original condition. Such causes in bridges are usually live loads, wind pressures, and temperature changes.

In the analysis of stresses and deflections of common truss systems the assumption is made that the elastic system when subjected to the specified causes undergoes deformations which are so small that the resulting distorted system may with sufficient accuracy be treated as undistorted and the original dimensions may be retained in the stress computations. This holds true for most stiff frames and the usual bridge trusses but it is not correct for long-span suspension bridges. The deformations of the original geometric figure

* *Proceedings*, Am. Soc. C. E., November, 1926, Papers and Discussions, p. 1771.

of suspension bridges are of such magnitude that serious differences in stresses result if the original dimensions are adhered to in the computations.

The reason for this phenomenon is that a suspension bridge or an arch, when erected, forms an elastic system in equilibrium under its dead load; that any live load or its equivalent causes a distortion of the original figure and establishes a new system in equilibrium. The main factors in the distortion are a change in length of the cable and a distortion of the stiffening trusses, likewise of the cable curve.

Prior to the connection of the individual truss members to form a continuous truss and prior to connecting the truss ends to their reaction points the cables form an equilibrium polygon subject to dead load concentrations as transmitted by the suspenders and the weight of the cable itself. After the connection is made an additional or live load on the system disturbs the old equilibrium and a new one tends to establish itself. This is accomplished by a deformation of the original polygon and truss. The truss deflects and this action is transmitted through the reactions of the suspenders to the cable. The latter thus forms a new polygon. The deformation is resisted by the elastic stiffness of the truss and by the work required to distort the original equilibrium polygon into the new one.

The stiffness of the truss is determined by its moment of inertia and the modulus of elasticity of its material. The stiffness of an equilibrium polygon on the other hand is a function of the forces acting upon it. The greater they are the more work will be required to distort the polygon. Therefore the greater the dead load of the bridge acting on the cable the stiffer the cable will be. The influence of the dead load on the stiffness of suspension bridges thus becomes apparent.

Let the following serve as an illustration by a rough example. Assume a span of 2 000 ft. with a versine of 250 ft. Such a span may be assumed to weigh 20 000 lb. per lin. ft. The horizontal pull in the cable will thus be $H = 40\,000\,000$ lb. Let a live load be placed on the bridge, which will cause a downward deflection at mid-span of, say, 5 ft. The newly distorted cable curve will thus be 5 ft. lower at its crown; in other words, its versine will be 255 ft. The new horizontal pull due to the dead load will become $H = 39\,216\,000$ lb., or about 784 000 lb. less. As is well known, the general expression for the bending moment at any point of the stiffening truss is given by the moment of the live load for a simple beam less the product of H times the ordinate of the cable curve at that point. Thus, multiplying the reduction in H (784 000 lb.) by 255 ft. gives a relieving moment of 200 000 000 ft-lb. which will reduce the bending in the truss. The effect of the live load also adds to the reduction. The magnitude of the reduction in the bending moment is a sufficient illustration of its importance both as to the stresses involved and as to the economy resulting.

From this example it becomes apparent that the heavier the bridge, the greater the reduction of the stresses in the truss will be. This emphasizes the common experience that the heavier a suspended chain, the more difficult it is to distort. The dead load effect was known, of course, to the early build-

ers of unstiffened suspension bridges, but was forgotten afterward by the introduction of the rigid and elastic theories.

The mathematical treatment consists in establishing the general equation of the curve of flexure of the truss and in deriving an expression of condition for the horizontal component of the cable pull, H, by equating the work of the external forces acting on the cable to its internal work of deformation. The expression for H is solved by successive numerical approximations. This is the time-consuming part of the computations. On the other hand, the determination of the moments and shears takes but little time. Moreover, the deflection of the truss at any point can be computed very readily. As explained, the entire theory is based on the deflection of the system. It has been thought appropriate, therefore, to name it the deflection theory.

The Manhattan Bridge, in New York, and the Delaware River Bridge, in Philadelphia, Pa., have been computed and designed in accordance with the deflection theory. In the case of the Delaware River Bridge, the speaker has made a comparison of the two theories by determining the bending moments and shears in the stiffening trusses for the identical structure. The results are given in Figs. 23, 24, 25, and 26, which show the moment and shear curves for the center and side spans plotted for both theories.

Quoting directly from the speaker's paper previously mentioned:

"The areas enclosed within the two curves represent the measure of the theoretical saving effected in the stiffening trusses. For the center span the application of the elastic theory would require theoretically 55 per cent. more material for the chords and 43 per cent. for the diagonal web members; for the side spans it would require 40 per cent. for the chords and 26 per cent. for the web members.

"In the actual design of the truss members certain minima sections had to be used for practical considerations. These reduce the theoretical saving to the actual saving of 51 per cent. for the chords of the central spans and 43 per cent. for its web members. The actual saving for the chords of the side spans is 38 per cent., and for the diagonal web members 24 per cent.

"The total saving in steel is 3 211 tons, or 42 per cent., of the total amount used in the trusses proper.

"Based on the contract prices obtained for the stiffening trusses, the cost of the 3 211 tons is:

Special steel...........	5 024 000 lb.	@ 8.75 c........	$440 000
Silicon steel...........	1 244 000 lb.	@ 7.85 c........	98 000
Carbon steel...........	154 000 lb.	@ 6.90 c........	10 000
Total			$548 000

The above sum of $548 000 represents the direct saving effected on the trusses.

"The increased weight of trusses would, however, require additional cable wire and suspender ropes to sustain it, which amounts to $206 000. Another $96 000 would be required by the towers and anchorage bents. A total of $302 000 would then be added to the saving on the trusses. The total saving effected by the use of the deflection theory would thus amount to $850 000."

In the Manhattan Bridge the reduction from the requirements of the elastic theory was to about 74% of the theoretical chord areas.

The speaker did not make any deductions as to the law of variation of one theory from the other or as to the percentage of reduction for various spans

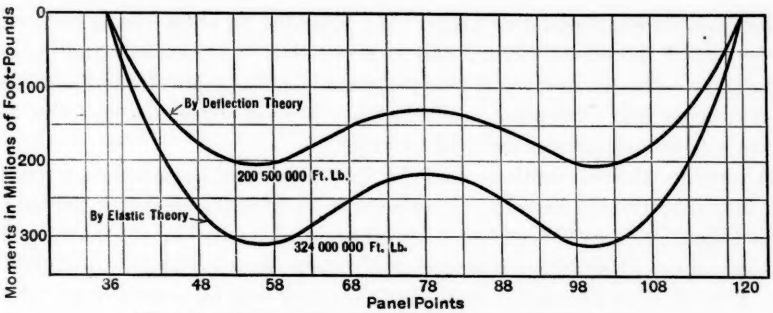


FIG. 23.—COMPARISON OF MOMENTS IN CENTER SPAN.

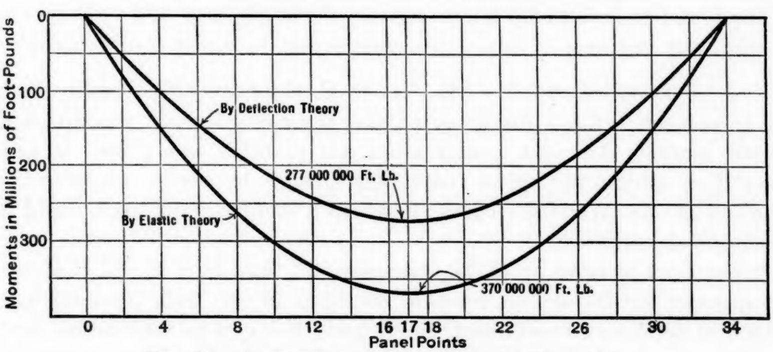


FIG. 24.—COMPARISON OF MOMENTS IN SIDE SPANS.

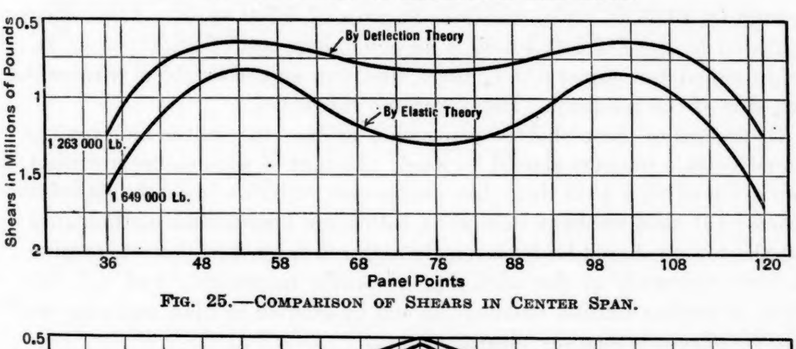


FIG. 25.—COMPARISON OF SHEARS IN CENTER SPAN.

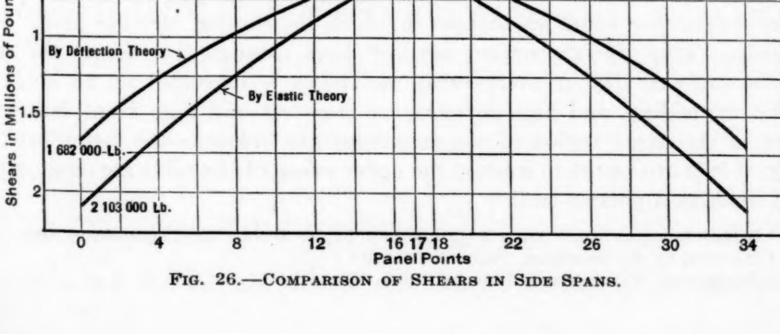


FIG. 26.—COMPARISON OF SHEARS IN SIDE SPANS.

under various dead and live loads. He intended to show the error involved in the use of the elastic theory and to prove that it is obsolete and should be abandoned. Mr. Waddell had evidently made his computations based on the elastic theory and afterward, realizing that it is in error, reduced the results by the single comparison available to him.

Altogether the assumptions made by the author and the results presented by him should be treated broadly. It is a question whether he ever intended them to be taken as rules for design. In other words, whether the author allows for the cables 70 000 lb. per sq. in., and others may go as far as 80 000 and even 100 000 lb., whether the same unit stresses will be used for nickel steel, or whether the assumed live load will persist, the general curves and figures will be found to be broadly approximate. Certainly they will be close enough for what any engineer of experience may expect to get in 2 or 3 hours' work.

V. A. Eberly,* Assoc. M. Am. Soc. C. E. (by letter).†—Designs for highway suspension bridges for spans of less than 500 ft. and proportioned for modern concrete floor-slab construction are probably not placed in bidding competition with other types frequently enough by bridge engineers. The extension of Mr. Waddell's curves down to a 250-ft. main span would prove of considerable interest.

It has been shown‡ by O. H. Ammann, M. Am. Soc. C. E., that suspension spans of 300 ft., or less, compete favorably in cost with truss spans. The writer has detailed a two-hinged, 250-ft. suspension span (the cables carrying no side spans) with a concrete floor-slab. The roadway slab measured 251.1 ft. long by 19.83 ft. wide, which is an area of 4 980 sq. ft. This bridge was estimated to cost $40 800, which is an equivalent cost of $8.20 per sq. ft. The design called for adjustable hangers, and two shop-assembled wire cables on each side of the roadway. There were no sidewalks.

In designing these shorter suspension bridges the writer believes (*a*) that non-adjustable hangers should be used; (*b*) that if wire cables are used, they should be carried back into the anchorages, eliminating the use of bridge sockets; (*c*) that eye-bars will prove better for long-period maintenance; (*d*) that the towers should be transversely battered inward at the top, to give more roadway clearance at the elevation of traffic movement; and (*e*) that the cables or eye-bar tension members should be cradled in plan and also to afford more satisfactory roadway clearances.

The embedded anchorages designed for the 250-ft. span suspension bridge differed from the usual anchorages in which the tension member passes in a sweeping curve over the upper part and down through the rear to the base. In the ordinary layout overturning resistance is depended on to hold the cables unyielding, and high forward toe pressures and high front face pressures in the upper region of the anchorage are induced—the latter particularly, if it is attempted to support the upper sweep of the cable arc on a narrow wall, such as a parapet wall.

* Bridge Designer, Ohio State Highway and Public Works Dept., Columbus, Ohio.

† Received by the Secretary, January 25, 1927.

‡ *Engineering News-Record*, June 21, 1923.

Under the method adopted by the writer (shown diagrammatically in Fig. 27 (*b*)) the entire anchorage, in order to yield, would move forward without rotating. Its resistance is derived from (1) bottom friction on its base; (2) friction on the sides; and (3) the resistance of a wedge-shaped section of earth as determined by the following consideration.

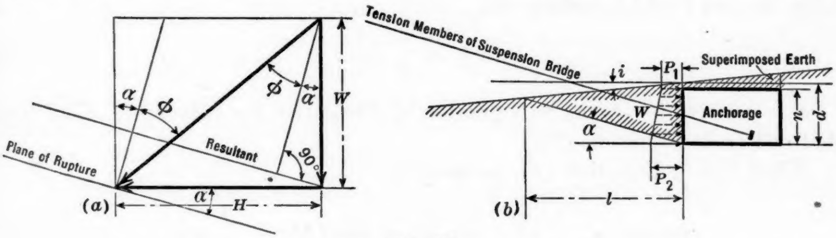


FIG. 27.

*Resistance of Earth in Front of a Retaining Wall to Sliding.**—In this deduction the cohesion of the earth is neglected, friction only being considered.

Let H = horizontal resistance to sliding of earth in front of wall of unit length.

w = unit weight of earth.

ϕ = friction angle of earth.

i = angle of slope of earth in front of wall.

d = depth of wall below surface of earth in front.

α = angle with horizontal of plane of least resistance to sliding.

W = total weight of earth in sliding wedge of unit length of wall.

Then, from Fig. 27 (*b*):

$$W = \frac{1}{2}\,w\,d\,l = \frac{\frac{1}{2}\,w\,d^2}{\tan\alpha + \tan i}$$

From Fig. 27 (*a*):

$$H\,(\cos\alpha - \sin\alpha\tan\phi) = W\,(\sin\alpha + \cos\alpha\tan\phi)$$

$$H = \frac{W\,(\tan\alpha + \tan\phi)}{1 - \tan\alpha\,.\,\tan\phi} = W\tan(\alpha + \phi)$$

$$= \frac{1}{2}\,w\,d^2\,\frac{\tan(\alpha+\phi)}{\tan\alpha + \tan i} \qquad \text{(2)}$$

In order to find the plane of least resistance to sliding, equate $\frac{\delta H}{\delta \alpha}$ (obtained by differentiating Equation (2) with H and α as variables), to zero; then,

$$\cos\alpha\sqrt{1-\cos^2\alpha} = \left(\cot\phi - \frac{\tan i}{2\sin\phi}\right)\cos^2\alpha - \frac{1}{2}\cot\phi \qquad \text{(3)}$$

Letting

$$A = \cot\phi - \frac{\tan i}{2\sin^2\phi}$$

* Deduced by A. W. Zesiger, M. Am. Soc. C. E., who has consented to this presentation.

Equation (3) reduces to:

$$\cos \alpha = \sqrt{\frac{1 + A \cot \phi}{2\,(1 + A^2)} \left[1 + \sqrt{1 - \frac{(1 + A^2) \cot^2 \phi}{(1 + A \cot \phi)^2}}\right]} \ldots\ldots\ldots (4)$$

In order to find for what value of i, $\alpha = 0$, let $\alpha = 0$ in Equation (3) and solve for tan i which reduces to,

$$\tan i = \frac{1}{2} \sin 2\,\phi \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

α is positive for values of i greater, and negative for values less, than that obtained from Equation (5).

When $i = 0$, Equation (4), reduces to,

$$\cos \alpha = \sqrt{\frac{1}{2}\,(1 + \sin \phi)} = \cos\left(45° - \frac{1}{2}\,\phi\right) \ldots\ldots\ldots\ldots (6)$$

or,

$$\alpha = 45 - \frac{1}{2}\,\phi$$

It also should be shown that H must not exceed the horizontal heaving power of the earth (Fig. 27(b)).

Let p_1 and p_2 represent the horizontal heaving power of the earth. Then the allowed H must not exceed $\frac{1}{2}\,n\,(p_1 + p_2)$.

The appearance of the suspension bridge stands foremost among steel bridges. The complete simplicity of the new self-suspending bridges at Pittsburgh, Pa., presents a bridge architecture of unequalled beauty. With a concrete-slab floor system the problem of the rigidity of the highway suspension bridge is solved to a great extent.

Horatio P. Van Cleve,* M. Am. Soc. C. E. (by letter).†—This interesting paper is in the hands of about 11 000 members of the Society, most of whom have never designed or estimated costs on a suspension bridge that has been actually constructed. Most of them, however, are interested in engineering costs as such, and in simple methods of arriving at the quantities from which those costs can be derived.

According to the Synopsis‡ of the paper: "Its object is to enable any engineer of good, general experience to compute, in two or three hours, * * * the total cost of any * * * suspension bridge * * *." For the purpose of preliminary estimate, as a means of comparison with other types of design, the desired object seems to have been attained, and the curves given should prove very convenient; but probably in many cases substructure and approach quantities would be subject to considerable modification if a closely approximate estimate of cost were desired. The same could probably be said of superstructure quantities, although to a less degree.

* Chf. Engr., J. Edward Ogden Co., New York, N. Y.

† Received by the Secretary, February 26, 1927.

‡ *Proceedings*, Am. Soc. C. E., November, 1926, Papers and Discussions, p. 1761.

Although engineering cost analyses are usually quite ephemeral and out of date soon after they are printed, the quantitative curves available in this paper should make it a fairly simple matter to revise the cost curves as conditions may dictate.

Rising costs of engineering erection labor have accompanied fairly uniform or falling prices of the chief construction materials (Fig. 28). Twenty years ago the average cost of structural steel shapes at Pittsburgh was $1.70. During the years of heavy war demand this cost rose rather sharply but now it is $1.95. This speaks volumes for the increased efficiency of manufacturing methods that have made such a condition possible in spite of the rising cost of labor. Since 1913, Portland cement has a little less than doubled in cost and is now falling somewhat, while Southern pine is double the price obtaining in 1913.

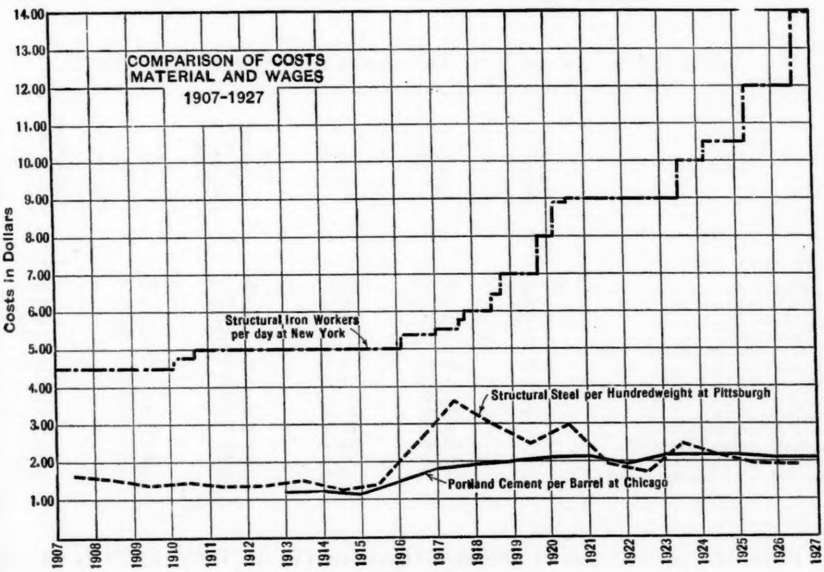


FIG. 28.

As to erection labor, however, it is a different story. In 1907 structural iron workers in New York City were receiving $4.50 per day. In 1927 the rate is $14.00. There is little or no increase in efficiency, however, and the day of decreased cost in this line does not seem to be in sight. Agitation for a 5-day week seems to be the vogue, and if it comes it will certainly mean a general increase in rates to offset the decrease in working time.

For iron-workers wages the increase during the last 10 years has been 16.4% per year (Fig. 28). This is exceeded by the concrete workers who have had an average increase of 18.3% per year over the 10-year period. It is probably fair to assume that the present cost of erection labor for average construction is not much less than 40% of the total cost. Assuming that erection wages increase for the next five years at the rate applying to the iron workers

of New York City over the 10-year period just ended, and that material prices remain as they are at present, each construction dollar, instead of being represented by $0.40 for erection and $0.60 for all other costs, will be composed of $0.55 erection cost and $0.45 for all other costs.

Of course, in some items of bridge construction the erection labor cost is far more than 40% of the total. This is particularly true of concrete piers. It hardly seems possible that erection labor can continue to advance at the average rate of the last ten years, for a general slowing down in building operations would result.

Assume that all erection labor were to increase at the rate of 16.4% per year, based on the present rates, and continued for five years. How would it affect the unit prices given in the paper? Table 4 is an attempt to separate erection costs from the totals given by the author, which are assumed to be correct, and to compare them with new units based on the assumed increase.

TABLE 4.—REVISED UNIT PRICES FOR ADVANCING LABOR COSTS.

Materials. (1)	Material delivered. (2)	Erection labor. (3)	Unit price. (4)	New erection labor Column (3) × Column (5). (5)	New unit price, Column (2) + Column (5). (6)	Percentage of increase, Column (6) over Column (4). (7)
Carbon steel	$0.045	$0.025	$0.07	$0.045	$0.09	28.6
Silicon steel	0.065	0.025	0.09	0.045	0.11	22.3
Nickel steel	0.075	0.025	0.10	0.045	0.12	20.0
Wire cables	0.15	0.03	0.18	0.055	0.205	13.9
Floor slab, including reinforcement	8.00	27.00	35.00	49.20	57.20	63.4
Pavement	1.60	1.20	3.00	2.18	3.78	26.0
Electric railway track	3.60	2.40	6.00	4.37	7.97	32.8
Shafts of piers	7.00	13.00	20.00	23.70	30.70	53.5
Shafts of anchorages	7.00	8.00	15.00	14.60	21.60	44.0
Bases of pneumatic piers	7.00	28.00	35.00	51.00	58.00	65.6
Bases of pile anchorages	7.00	23.00	30.00	41.90	48.90	63.0
Piles below anchorage bases	.25	1.00	1.25	1.82	2.07	65.5

From the percentages of increase (Column (7) of Table 4), it is evident that advancing costs of erection labor are a matter of considerable moment to the engineer and contractor as well as to the general public.

The matter appearing in the paper is new, concise, and much to the point. It has involved a great deal of painstaking labor and a real service to the profession has been accomplished.

WENDELL P. BROWN,* M. AM. SOC. C. E. (by letter).†—This paper is a most valuable contribution to engineering data. Few bridge engineers have the opportunity in the regular course of their professional activities to determine the cost of such long-span structures. Therefore, these data will be valuable as a reference and check on similar estimates for those who are required to make them.

* Pres. and Chf. Engr., The Wendell P. Brown Co., Cleveland, Ohio.

† Received by the Secretary, February 26, 1927.

The use of unit costs per square foot of floor of structure for preliminary estimates is a satisfactory and fairly accurate method, but it should be done by those who are familiar with the use of such data, and experienced in their application to the various conditions affecting the particular estimate under consideration. Although not in a position to confirm the estimates for structures of such unusual length of span, the writer judges from the unit prices used for the different classes of materials that costs per square foot are conservative and sufficient.

The cost of lighting, however, seems rather low, except for a very simple type of installation, but this item is a small proportion of the entire cost and the total would be but little affected. The use of tower encasement, while permissible, and perhaps desirable under certain conditions, is of somewhat questionable value, either from the standpoint of appearance or of practicability.

VENTURI TUBE CHARACTERISTICS

Discussion*

By Messrs. J. E. Gibson, D. E. Davis, Joseph A. Russell, W. S. Pardoe, Bayard F. Snow, and Morrough P. O'Brien.

J. E. Gibson,† M. Am. Soc. C. E. (by letter).‡—It was the writer's great privilege to be associated with the author from 1910 to 1917 during that period when the Simplex Venturi tube and registering mechanism was being developed, and to take part in the determination of the Venturi tube characteristics; and thus to become familiar with the development of the various means of measuring the Venturi differential heads. It was particularly gratifying to obtain the results of the hook-gauge method which added immensely to the accuracy of these determinations, and to learn from these experiments that the Venturi tube ratio could be materially reduced without sacrificing accuracy. Previous to this, it was the practice to make the Venturi tube ratio 9 to 1, in order to secure satisfactory accuracy, and this large ratio caused a material constant loss of head through the Venturi tube. For periodic measurements, this loss of head is not material, but for continuous records it is a serious objection. The reduction in the Venturi ratio to as low as 4 to 1 (or even lower) constituted a decided step forward in the art of fluid measurement. The nearer the area of the throat to that of the main pipe line, the more efficient becomes the pipe line for utility service, particularly fire service.

The author should be complimented on the completeness and scope of the tests as this series is by far the most complete and useful data on Venturi tubes that have been heretofore published to the writer's knowledge.

The results shown by the test of the tuberculated Venturi tube were particularly impressive. No water department is satisfied without a continuous record of the water being delivered by the plant, and while the results on new tubes are very valuable, the feature that most interests the operating man is the continuing accuracy of the meter after several years of operation when the tube and mains become tuberculated. These results show that while the loss of head through the tube may be increased as much as four or five times, the effect on the coefficient does not exceed 2½ per cent. In other words, it means that loss in accuracy of an old tube and meter, due to fouling, is less than 3%, or within the limits of observation, placing the Venturi tube as a measuring instrument superior to any other known method.

* This discussion (of the paper by J. W. Ledoux, M. Am. Soc. C. E., published in November, 1926, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Mgr. and Engr., Commrs. of Public Works, Charleston, S. C.

‡ Received by the Secretary, December 9, 1926.

D. E. Davis,* M. Am. Soc. C. E. (by letter).†—That Mr. Ledoux's paper is an exceedingly valuable and useful contribution to hydraulic literature will be especially appreciated by those who have had occasion to measure large quantities of water in connection with testing of pumps, etc., where great accuracy is at a premium. The care with which the water was weighed or measured volumetrically, and particularly the accuracy with which the differential heads on the Venturi tube were observed, render these tests most authoritative and inspire entire confidence in the results.

It would appear that the coefficients found in these experiments may be applied with assurance to any well-designed Venturi tube of similar diameter for throats of 3-in. or more. For smaller throats, variations may be found in individual tubes, which might develop some deviation from the coefficients found in these experiments.

The effect of deposits within the tube itself on the coefficients of discharge, is well brought out in Fig. 1.‡ It emphasizes the necessity of a clean tube where accurate tests are to be undertaken. Most observers are familiar with similar effects in ordinary operation where proper precautions in the blowing out of mud rings and the release of air have not been taken.

The writer has been interested in comparing the results of Mr. Ledoux's latest tests with the curves of coefficients shown in the report published by the American Society of Mechanical Engineers.§ Each recognizes that the throat velocities substantially determine the coefficients; and that the diameters of the pipe proper and the form of the tube exert relatively little effect. Each also recognizes increasing coefficients with increasing velocities, especially up to speeds of 10 ft. per sec. Beyond that point the curves shown in the report tend to rise slowly, whereas those of the paper approach a constant figure. Mr. Ledoux shows consistently lower coefficients. A comparison between a 12-in. and a 1-in. throat is shown in Table 6.

TABLE 6.—Comparison of Venturi Discharge Coefficients.

Authority.	Throat diameter, in inches.	Throat Velocities, in Feet per Second.			
		4	10	20	30
Author..........	12	0.977	0.980	0.982	0.982
"Fluid Meters"..	12	0.983	0.987	0.989	0.991
Author..........	1	0.945	0.960	0.965	.....
"Fluid Meters"..	1	0.966	0.974	0.978	0.981

The explanation of the differences may possibly be found in Mr. Ledoux's experience with vortex action, which tends to give incorrect readings for the differential head on the meter itself. It is understood that the curves in the report on "Fluid Meters" were derived from formulas, the empirical coeffi-

* (The J. N. Chester Engineers), Pittsburgh, Pa.

† Received by the Secretary, January 14, 1927.

‡ *Proceedings*, Am. Soc. C. E., November, 1926, Papers and Discussions, p. 1793

§ "Fluid Meters", Pt. 1, 1924 Edition.

cients of which depend on available experiments heretofore conducted, and it is possible that inaccuracies may have crept into some of these experiments due to head readings, vortex action, etc., which may account for coefficients higher than those of Mr. Ledoux's experiments.

The writer had occasion to observe the excessive influence of vortex action where flows were being checked by pitometer tubes set at right angles near the center of the stream. The alternate rise and fall of the fluid in the two manometers pointed to a vortex action and a volumetric check of water as measured in a basin fully corroborated the suspicions, for a variation of several per cent. was conclusively established between various runs, depending on the manner in which the vortex action was set up.

In a recent check on a Simplex Venturi meter, the dimensions of which were 36 by 18 in., the writer had occasion to make four volumetric checks in a basin where accurate hook-gauge observations could be made. The throat velocities were about 20 ft. per sec. and the average coefficient was found to be 0.983, which is identical with the results in Mr. Ledoux's Test No. 25-A for a 30 by 18-in. tube (Table 5*). In these checks the maximum variation was 0.003 in terms of the average coefficient.

The author's conclusion, that with the proper choice of coefficients the Venturi tube is an extremely accurate measuring device, is fully supported, and great credit for the experiments which have contributed to this accuracy is due Mr. Ledoux.

JOSEPH A. RUSSELL,† ASSOC. M. AM. SOC. C. E. (by letter).‡—The author has presented a most interesting paper and one which deserves careful reading. The tests on which it was based might at one time have been considered of interest only for a textbook on hydraulics or to a designer of metering devices, but in the preparation of the tables and in his comments on the data collected, Mr. Ledoux has made it plain that the tests are of importance to engineers in very much broader fields. In addition, the tests and the brief summaries thereof correct certain mistaken ideas formed by undergraduates.

The author's conclusions as to the design of tubes, the loss of head as affected by design, and the uniformity of accuracy for wide variations in size are important to the designer, purchaser, or user of Venturi tube equipment. The designer may obviously take more liberties to obtain the desired objectives without affecting the accuracy. The purchaser or user is provided with definite data by which to compare the accuracy of different sizes and ratios of Venturi tubes.

The series of tests reported in Table 4§ are impressive. Some engineers have questioned after experiencing loss of accuracy in Venturi meters by tuberculation in the throat and mouth sections of the old, all-iron, Venturi tubes, whether better results would not be assured by providing a complete bronze up-stream section and throat. It seems apparent from this series of tests that the adequacy and extra expense for bronze throat and mouth con-

* *Proceedings*, Am. Soc. C. E., November, 1926, Papers and Discussions, p. 1795.
† Engr., Penna. R. R. Water Cos., Philadelphia, Pa.
‡ Received by the Secretary, January 19, 1927.
§ *Proceedings*, Am. Soc. C. E., November, 1926, Papers and Discussions, p. 1793.

nections are substantiated. Like many other things, however, satisfaction as regards continued accuracy is diminished somewhat by observing the loss of head for corresponding quantities, as reported in the last column of Table 4 on the runs before and after cleaning.

The writer is impressed by Mr. Ledoux's remarks as to the accuracy of the tube. As a user of this type of equipment, it is comforting to know all these important characteristics and that the manufacturers and designers go to such lengths and expense, as is evidenced by this series of tests, to substantiate calculation and design. In some cases it may make little difference to the user whether the coefficient of discharge of his Venturi tube is 0.930 or 0.970, but he should be quite concerned as to whether the designer knows this and whether or not he has accommodated his equipment to this fact.

W. S. PARDOE,* M. AM. SOC. C. E. (by letter).†—This paper places before engineers data that are usually considered to be private property by firms manufacturing inferential meters.

The writer has published‡ the following expression, with its derivation, for the coefficient of Venturi meters,

$$C = \frac{\sqrt{1 - \left(\frac{d_2}{d_1}\right)^4}}{\sqrt{1 - \left(\frac{d_2}{d_1}\right)^4 + 5.75 \left\{ f_1 \frac{l_1}{d_1} \left(\frac{d_2}{d_1}\right)^4 + f \frac{m^5}{4\,d_2} \left(\frac{1}{m^4} - \frac{1}{(m+l)^4}\right) + f_2 \frac{l_2}{d_2} \right\}^{1.25}}}$$

f_1, f, and f_2 are the coefficients of friction for clean cast-iron pipe used by the late Hamilton Smith, M. Am. Soc. C. E. The other notation is shown on Fig. 5.

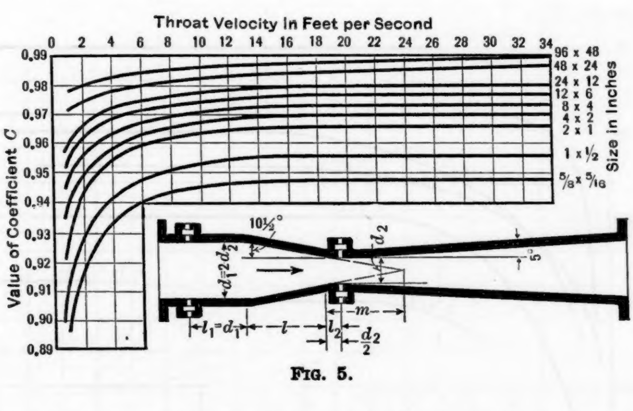


FIG. 5.

The formula was obtained semi-empirically from data of tests of Venturi meters from $\frac{5}{8}$ by $\frac{7}{16}$ in., to 16 by 8 in., and from tests of a 6-in., brass, conical nozzle with various sized tips. The close agreement of this

* Prof. of Hydr. Eng., Univ. of Pennsylvania, Philadelphia, Pa.

† Received by the Secretary January 5, 1927.

‡ *Engineering News-Record*, September 25, 1919.

formula and experimental results is shown by Fig. 6 for a 4 by 2-in. Venturi meter (Builders Iron Foundry) and by comparing the curve in Fig. 5 for the 12 by 6-in. meter with Figs. 2 * and 3 † in the paper. Recent tests on larger meters have not indicated a necessity for changing the formula, but have rather confirmed it.

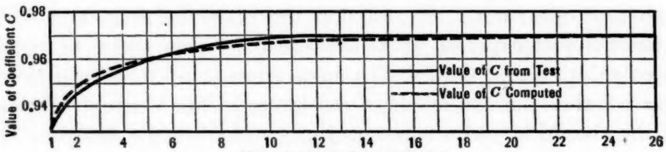


FIG. 6.

Tests of a 4 by 2-in. Simplex Venturi meter obtained from H. P. Hammond, M. Am. Soc. C. E., at the Polytechnic Institute, Brooklyn, N. Y., give an average value of C of 0.971 from 10 to 50 ft. per sec. The tests were made volumetrically and confirm tests of Simplex Venturi meters in the Hydraulic Laboratory of the University of Pennsylvania.

The writer found such a close agreement between the 4 by 2-in. Simplex Venturi meter and the 4 by 2-in. Builders Iron Foundry Venturi meter in the University of Pennsylvania Laboratory that he has collected coefficient curves of several Builders Iron Foundry Venturi meters tested in reliable college laboratories. These are shown in Fig. 7 and were obtained from the following sources:

(*a*) 36 by 16-in. meter at Worcester Polytechnic Institute, C. M. Allen, M. Am. Soc. C. E.

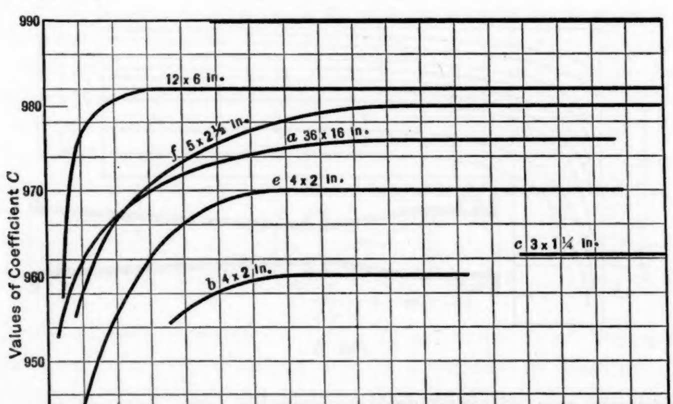


FIG. 7.—COEFFICIENTS OF BUILDERS IRON FOUNDRY VENTURI METERS AT VARIOUS ENGINEERING INSTITUTIONS.

* *Proceedings*, Am. Soc. C. E., November, 1926, Papers and Discussions, p. 1794.
† *Loc. cit.*, p. 1795.

(*b*) 4 by 2-in. meter at The Harvard Engineering School, H. J. Hughes, M. Am. Soc. C. E.

(*c*) 2.94 by 1.25-in. meter at University of Toronto, Applied Science Department, Professor Robert W. Angus.

(*d*) 30 by 15-in. meter at Massachusetts Institute of Technology, George E. Russell, M. Am. Soc. C. E.

(*e*) 4 by 2-in. meter in University of Pennsylvania Civil Engineering Laboratory.

(*f*) 5 by $2\frac{1}{2}$-in. meter in University of Pennsylvania Mechanical Engineering Laboratory.

Curves *a, b, c,* and *d* (Fig. 7) are about what should be expected, but Curve *d* is very low. The discharge in this case was measured by a 10-ft. suppressed weir; probably the error does not lie in this fact but rather in that the meter is placed shortly after a large side-suction centrifugal pump which discharges the water in the form of a free vortex. The result is a lowering of the coefficient, as the tangential velocity varies inversely as the radius; that is, v varies as $\frac{1}{r}$. Consequently the Venturi head would be greater than that due to the axial velocity only; hence the reduction in coefficient. That this is possible is shown by Fig. 8 giving tests on a small brass Venturi meter, using free vortices of various angles.

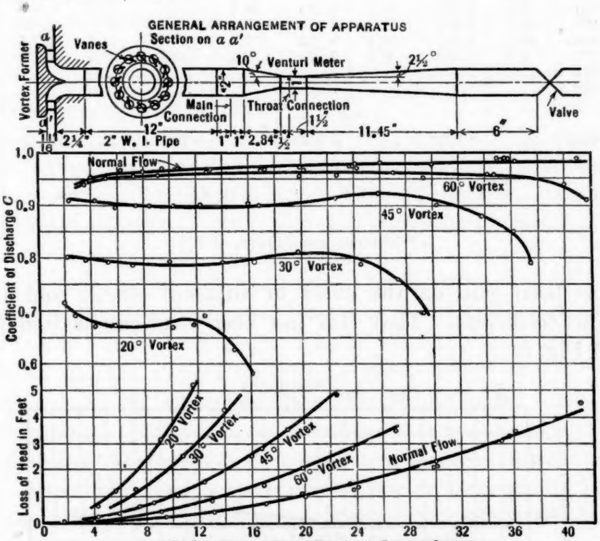


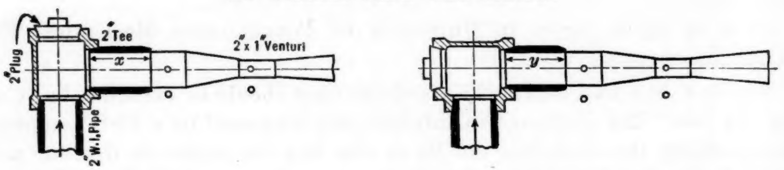
Fig. 8.—Tests of 2 by 1-Inch Venturi Meter Showing the Effect of Passing Water Through It in Form of a Free Vortex.

The writer is aware of the discussion * on this matter by Mr. A. H. Gibson, of Manchester, England, in which helical flow or a forced vortex is considered and an increase in coefficient is obtained. This the writer has also confirmed

* Abstracted in Pt. I, Report of Special Research Committee of Am. Soc. Mech Engrs., on Fluid Meters.

by experiment. Hence, a free vortex will decrease the coefficient and a forced vortex (v varies as r) will increase it. Incidentally, he believes the free vortex more likely to form and persist in a straight pipe than a forced vortex with high tangential velocity at the wall of the pipe.

TABLE 7.—TEST OF 2 BY 1-INCH VENTURI METER, SHOWING THE EFFECT OF PLACING A 2-INCH TEE AT VARIOUS DISTANCES UP STREAM FROM THE METER.



x = 2 in.		x = 4 in.		x = 6 in.		x = 10 in.		y = 2 in.		y = 4 in.	
V_t.	c.	V_t.	c.	V_t.	c.	V_t.	c.	V_t.	c.	V_t.	c.
45.6	0.9805	45.3	0.987	45.3	0.985	45.2	0.981	41.4	0.974	44.2	0.971
41.5	0.977	37.2	0.984	41.2	0.983	41.2	0.979	39.0	0.975	41.4	0.985
37.3	0.973	26.6	0.979	37.0	0.977	36.9	0.978	34.8	0.975	37.4	0.984
32.3	0.972	23.4	0.980	32.2	0.979	32.1	0.9755	29.4	0.973	32.5	0.982
29.5	0.9715	21.4	0.977	26.3	0.974	26.6	0.9765	26.5	0.9755	26.7	0.981
26.6	0.972	19.4	0.976	23.1	0.973	23.3	0.9765	23.2	0.9755	23.5	0.980
23.3	0.971	17.5	0.9785	19.3	0.974	19.3	0.977	19.5	0.978	19.5	0.980
21.4	0.970	13.7	0.975	17.2	0.976	17.4	0.975	17.0	0.9735	17.1	0.980
19.4	0.9685	10.2	0.971	13.4	0.975	13.6	0.972	13.5	0.976	13.5	0.976
17.4	0.977	8.9	0.969	8.3	0.973	8.6	0.969	9.4	0.969	8.7	0.972
13.8	0.976	....	......	....	......	....	......	....	......	....	......
9.0	0.972	....	......	....	......	....	......	....	......	....	......
Average..	0.9739	....	0.9776	....	0.9769	....	0.9760	....	0.974	....	0.9791
Low*............		O. K.		O. K.		O. K.		Low		Close	

* Normal coefficient, 0.977.

Much has been said on the effect of adjacent elbows and partly closed valves on the coefficient. That this has been over-emphasized is shown by Table 7 and Fig. 9.

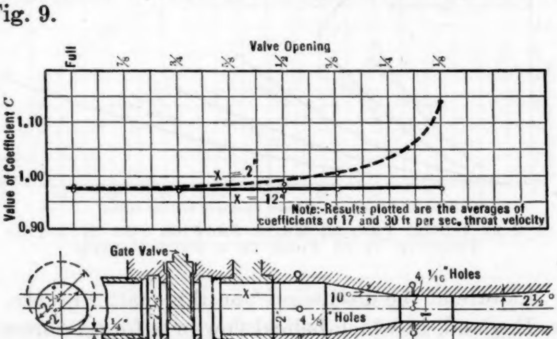


FIG. 9.—SHOWING EFFECT OF PARTLY CLOSED VALVES.

In all cases when the coefficient has appeared to be abnormal it has been due to a vortex, either forced or free, and the meters could be made to give correct coefficients by means of a straightening vane in their up-stream sections.

Mr. Ledoux arrives at the conclusion that the coefficient is 0.977 maximum, and increases with the velocity. The Builders Iron Foundry, as far as the writer knows, uses 0.99 for all sizes and velocities within the range of its recording device. The writer believes that the coefficient varies both with velocity and size of meter, although there are tests which may not confirm this.

The Venturi meter is much more accurate than the weir and slightly less accurate than the orifice. Its chief advantage over the orifice lies in the fact that the loss of head in the Venturi meter is about 25% that in the orifice meter for the same Venturi head.

BAYARD F. SNOW,* ASSOC. M. AM. SOC. C. E. (by letter).†—The correct measurement of liquids is of such importance to hydraulic engineers that all should welcome any data that contribute to the accuracy and refinement with which measurements may be made and interpreted. The question, however, of whether 0.96 or 0.985 should be used as the coefficient in a given case, although apparently of considerable importance, is occasionally of relatively minor interest.

The writer has recently superintended the change of a Venturi meter tube on a sewage discharge main to prepare for the larger flows anticipated under new conditions. The old tube had been in use for twenty years. Although it was known that industrial wastes built heavy deposits in the grade sewers tributary to the pumping station, no one had thought that such formations would occur in the throat. Pumpage rates of 11 000 000 to 11 500 000 gal. per day were apparently the best that could be obtained with the present equipment, but as this meant a throat velocity of 14 ft. per sec., naturally extensive deposits were not anticipated.

When the throat section was removed a thick formation of trade wastes, consisting principally of carbonate of lime, was noticed. The condition of the meter as a whole is shown on Fig. 10. The 15-in. throat had been reduced to a smooth surfaced passage almost circular in cross-section and only 12¼ in. in diameter. The deposits up stream from the throat were thicker and more irregular. Computations based on these measurements indicated that the meter had been registering more than 50% in excess of the actual quantities pumped, or expressed differently, that a correction factor of 0.63 should be applied to recent readings. No better information being available, it has been assumed that this correction factor would decrease uniformly from 1.0 during the time the meter tube had been in service. When the deposits were removed from the part of the tube to remain in service, it was noticed that the tar coating was in perfect condition under this protection. After all changes had been completed, the register indicated that the pumpage, which under identical conditions had formerly been measured as 11 000 000 to 11 500 000 gal. per day now showed

* Res. Engr., South Essex Sewerage Dist., Salem, Mass.

† Received by the Secretary, January 18, 1927.

as 7 000 000 to 7 250 000 gal., thus checking the computed correction factor. This change was made in April, 1926, and recent measurements have shown deposits approximately 0.1 in. thick, of a consistency similar to that of smoothly troweled cement mortar 2 or 3 hours old.

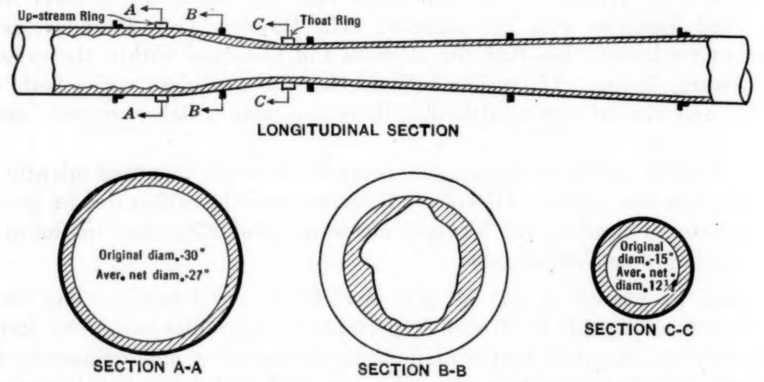


FIG. 10.—VENTURI METER SECTIONS, SALEM, MASS., SEWAGE PUMPING STATION.

In this problem, the question is not should a factor of 0.96 or 0.985 be used, but can the areas at the piezometer rings be maintained within reasonable limits of the original; in other words, can the Venturi tube be depended on for measurement of sewage and trade wastes?

MORROUGH P. O'BRIEN,* JUN. AM. SOC. C. E. (by letter).†—The geometric similarity of several groups of the tubes listed in Table 1‡ gives an opportunity to check the laws of dynamic similarity. Although the tubes were probably not of as exact scale as would be used in model tests, nevertheless the results are in fair agreement with the theory. It would have helped greatly in extending the theory if other liquids had been used in addition to water and if the water temperatures had been given.

It is usually assumed that the head loss in any sort of closed pipe varies with the velocity (V), the diameter (d), and the acceleration of gravity (g). Expressing this relation in a general form,

$$f \propto V^x \, d^y \, g^z$$

in which x, y, and z are numerical coefficients. Applying the theory of dimensions,§

$$L = L^x \, T^{-x} \, L^y \, L^z \, T^{-2z}$$

$$\left.\begin{array}{l} 1 = x + y + z \\ 0 = -x - 2z \end{array}\right\} \begin{array}{l} x = -2z \\ y = 1 + z \end{array}$$

and,

$$f \propto \frac{d^{(1+z)} \, g^z}{V^{2z}}$$

* Research Asst., Eng. Experiment Station, Purdue Univ., Lafayette, Ind.
† Received by the Secretary February 14, 1927.
‡ *Proceedings*, Am. Soc. C. E., November, 1926, Papers and Discussions, p. 1790.
§ "Mechanical Properties of Fluids," D. Van Nostrand, N. Y., p. 185.

or,

$$\frac{f}{D} \propto \left(\frac{d\ g}{V^2}\right)^z$$

According to the theory of dynamic similarity, if $\frac{f}{d}$ is the same for two geometrically similar tubes, $\left(\frac{d\ g}{V^2}\right)$ also has the same value for both tubes. Therefore, since g varies but little, the conditions for similarity are that (1) f varies with D; and (2) f and D vary with V^2. That Venturi tubes do not exactly fulfill the second requirement is shown by the values of n in Table 1, but as these values have an average value of 2 in Mr. Ledoux's general formula,

$$f = \frac{V_1^{\ 2}}{470}$$

the characteristics of groups of geometrically similar tubes should show an agreement equal to the degree of approximation of this formula. In Figs. 11, 12, and 13, values of $\frac{f}{d}$ are plotted against $\frac{V}{\sqrt{d}}$ which is equivalent to reducing the head loss and velocity to values corresponding to a throat diameter of 1 in. For equal values of $\frac{V}{\sqrt{d}}$ the corresponding values of $\frac{f}{d}$ seem to be greater for the smaller sized tubes, showing a scale effect which would have to be taken into account in predicting the loss of head in a large Venturi tube. However, in Fig. 12 the curves for the 36-in. and 2-in. tubes lie between those for the 16-in. and 4-in. tubes.

In the general formula for the loss of head, the viscosity of the liquid was not considered. Including the kinematic viscosity, K, in the general expression,

$$f \propto V^x\ d^y\ g^z\ K^s$$

where x, y, z and s are unknown numerical coefficients, the relation becomes,

$$\frac{f}{d} \propto \left(\frac{D\ g}{V^2}\right)^z \left(\frac{D\ K}{V^2}\right)^s$$

The values of s and z are to be determined experimentally, but as the temperatures are not given and as the general formulas do not include d as a variable they cannot be evaluated from the values given in Table 1. The viscosity undoubtedly accounts for the scale effect, but as its influence decreases with increasing turbulence the agreement should be best at high throat velocities. This is substantiated by Figs. 11, 12, and 13, in which, with the exception of Tube No. 8, the lines seem to be converging for higher values of $\frac{V}{\sqrt{D}}$.

The quantity discharged through a Venturi meter depends on the area of the throat section, A, the operating head, h, the acceleration of gravity, g, and the kinematic viscosity, K. Writing a general equation for this relation,

$$Q \propto A^x\ h^y\ g^z\ K^s$$

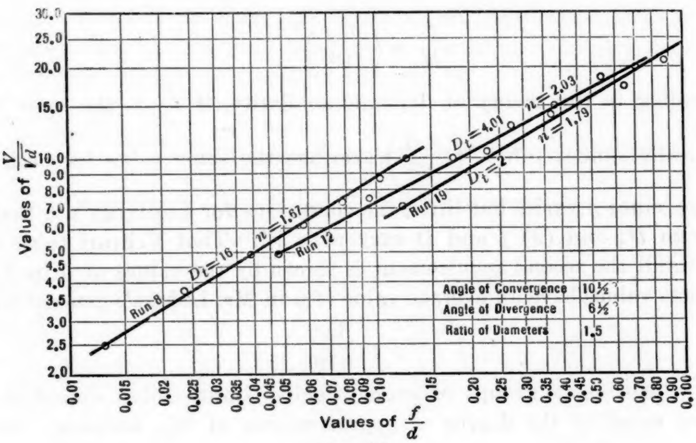


FIG. 11.

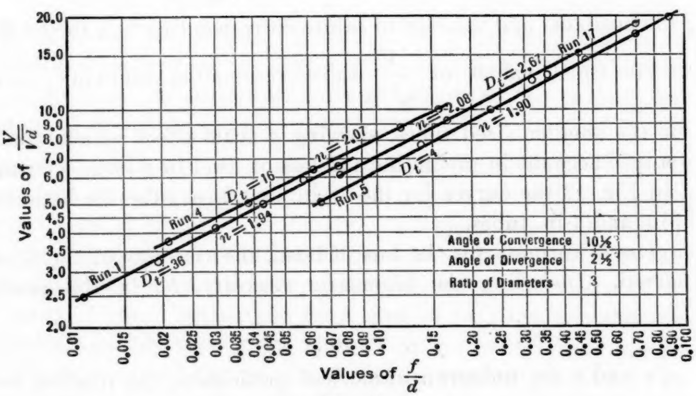


FIG. 12.

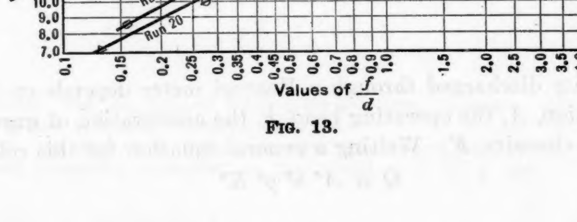


FIG. 13.

and applying the principle of dimensional homogeneity, the equation in terms of the fundamental dimensions becomes,

$$L^3\, T^{-1} \propto L^{2x}\, L^y\, L^z\, T^{-2z}\, L^{2s}\, T^{-s}$$

Adding powers of the same dimension,

$$3 = 2\,x + y + z + 2\,s$$
$$-1 = -2\,z - s$$

and the original equation becomes,

$$\frac{Q}{g\ h^{\frac{5}{2}}} \propto \left(\frac{A}{h^2}\right)^x \left(\frac{K}{g^{\frac{1}{2}}\ h^{\frac{3}{2}}}\right)^s$$

The expression, $\left(\frac{A}{h^2}\right)$, is the same for all geometrically similar tubes and $\sqrt{g\ h}$ may be replaced for V, so that,

$$\frac{Q}{A\ \sqrt{g\ h}} \propto \left(\frac{K}{V\ h}\right)^s$$

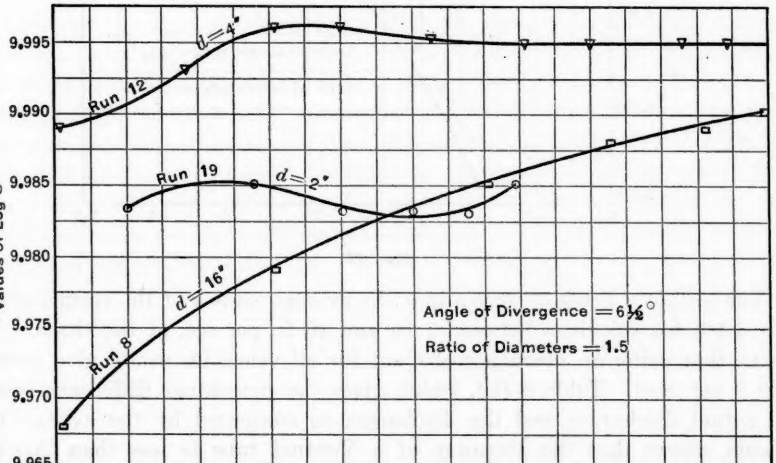


FIG. 14.

If the heads are in proportion to the other linear dimensions, h may be replaced by d, then,

$$\frac{Q}{A\ \sqrt{g\ h}} \propto \left(\frac{K}{V\ d}\right)^s$$

As $\frac{Q}{A\ \sqrt{g\ h}}$ is proportional to C, equal values of $\left(\frac{K}{V\ d}\right)$ should give equal values of C in all geometrically similar tubes.* If K is a constant, equal values of ($V\,d$) should give equal values of C. Figs. 14 and 15 show such points for Runs 8, 9, 12, 19, and 25 A. Although the differences are greatly exaggerated

* "Hydraulics and Its Applications," by A. H. Gibson, D. Van Nostrand Co., N. Y., p. 724; Report on Fluid Meters, Am. Soc. Mech. Engrs.

by the scale, there is fair agreement between the curves which tend to converge for the higher values of $V\,d$. The kinematic viscosity varies 35% between 50 and 75° Fahr. and 10% between 60 and 70° Fahr., and it seems certain that some of the discrepancy would be eliminated by taking the water temperatures into account. If liquids of varying viscosities were run through these same tubes, curves of $\left(\frac{K}{V\,d}\right)$ and C could be obtained for each series of similar tubes. This is the method which Osborne Reynolds and Stanton and Pannell used in their studies of similarity in circular pipes.

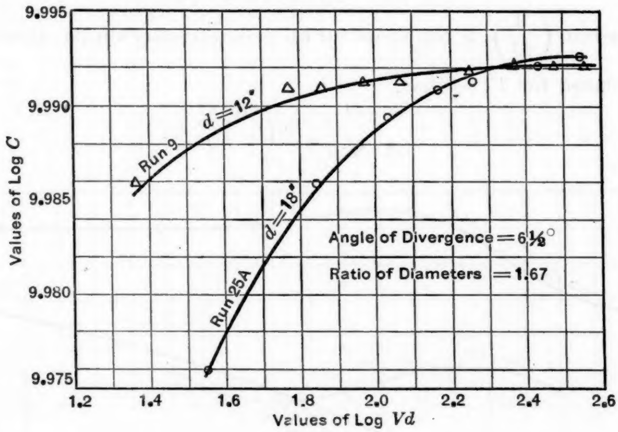


FIG. 15.

The author's Table 3* shows that the average values of the coefficient vary only 4.4% for velocities between 1 ft. and 30 ft. per sec. at the throat. This means that using an average coefficient for all velocities would give errors of only 2 per cent. Table 3 (*b*), which gives the percentage difference between the actual discharges and the discharges as computed by the average coefficients, shows that the accuracy of a Venturi tube is less than that of a carefully constructed triangular weir.

In Table 8 the values of n and a, for the different groups of similar tubes, show a rather astonishing variation. They do not seem to vary with the throat diameters, the ratio of diameters, or the angle of divergence; the only apparent relationship is that high values of a are accompanied by high values of n as would be expected. The usual assumption in regard to an equation expressing a relationship between measured quantities is that it will be a continuous function and the only continuous function which would relate n, a, and d would be periodic. Studies of pipes, orifices, and weirs do not show any reason for believing this to be the case, since the throat velocities were above the unstable region between sinuous and non-sinuous flow.

Whatever may be the cause of the variations in n, a, and c, it is evident that the Venturi tube is extremely sensitive to small changes in construction, and it seems necessary that each tube should be calibrated if an

* *Proceedings*, Am. Soc. C. E., November, 1926, Papers and Discussions, p. 1792.

accuracy better than 1% is desired. It would be interesting to compare the performance of two commercial tubes of identical dimensions.

TABLE 8.—VALUES OF n AND a FOR DIFFERENT GROUPS OF SIMILAR TUBES.

Run No.	Ratio, R.	Throat diameter, d.	n.	a.	Angle of divergence, in degrees.
19	1.50	2.	1.79	248	6½
12	1.47	4.01	2.03	636	6½
8	1.50	16.	1.67	242	6½
23	2.01	1.	1.75	181	2½
20	1.99	2.	1.92	306	2½
15	2.	3.	1.93	388	2½
3	2.13	22.5	2.03	698	2½
2	2.11	22.5	2.02	453	2½
17	3.	2.67	2.08	603	2½
5	3.	4.	1.90	303	2½
4	3.	16.	2.07	804	2½
1	3.	36.	1.94	477	2½

Although the tests given in Table 8 do not greatly improve the status of model testing as a means of studying the performance of large-sized structures, they bring out no serious contradiction of the theory. If Tests 5 and 17 had been used to predict the performance of Tube No. 1 (Fig. 12), the error would have been less than 10% in the discharge or loss of head; and if a previous study had shown the approximate amount of "scale effect", the error would have been much less. While a 10% error would not be permissible for a Venturi tube, it would be allowable in the case of other hydraulic structures where it is impossible to test the full-scale prototype.

STORAGE REQUIRED FOR THE REGULATION OF STREAM FLOW

Discussion*

BY MESSRS EDWARD H. SARGENT, ALLEN HAZEN, ROGER W. ARMSTRONG, H. ALDEN FOSTER, JOEL D. JUSTIN, L. STANDISH HALL, AND EDWARD M. CRAIG, JR.

EDWARD H SARGENT,† M. AM. SOC. C. E. (by letter).‡—Mr. Sudler has earned the thanks of the profession for this paper summarizing his studies of the storage required for stream-flow regulation.

For about ten years the writer was in charge of the investigations of the water storage and power possibilities of New York State made by the Conservation Commission. Most of the results of his studies were embodied in the Annual Reports of that Commission for the years 1911 to 1921. Attention is invited particularly to the 1919 Report in which is given a set of duration curves of most of the streams of the State and also curves showing the relation between duration curves plotted from daily and monthly flow values, respectively.

Herewith are submitted two diagrams (Fig. 34 (*a*) and Fig. 34 (*b*)) relating to stream flow regulation, typical of many similar diagrams, which were not published. Fig. 34 (*a*) shows the regulation that could have been maintained 100% of the time, that is, continuously, on various streams with given amounts of storage. These curves were plotted from data obtained from mass and duration curve studies of the streams in question. Fig. 34 (*b*) is similar to Fig. 34 (*a*), except that it shows the flows that could have been maintained 60% of the time for given amounts of storage.

It was the writer's intention to prepare and publish in the reports of the Conservation Commission similar curves for all streams in the State of which there were reliable stream-gauging records; but the decision of the Legislature to cease making appropriations for water power investigations prevented this. It was believed that these curves, used in conjunction with comparative hydrological and topographical data, would have afforded a quick means of making storage and regulation studies.

The writer's thoughts as to stream regulation for the combined purposes of power development and flood control were given in his discussion§ of the paper by J. C. Stevens, M. Am. Soc. C. E., entitled "Stream Regulation with

* This discussion (of the paper by Charles E. Sudler, M. Am. Soc. C. E., published in December, 1926, *Proceedings* and presented at the meeting of February 2, 1927), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Chf. Engr., Hudson River Regulating Dist., Albany, N. Y.

‡ Received by the Secretary, December 21, 1926.

§ *Proceedings*, Am. Soc. C. E., May, 1926, Papers and Discussions, p. 1036.

Reference to Irrigation and Power",* as were also his studies of storage reservoir operation. Subsequent studies along this line have led to the belief that careful examination of the hydrology of a water-shed will make it possible in the actual operation of a reservoir to get from 85 to 90% of the so-called "ideal" or theoretical regulation.

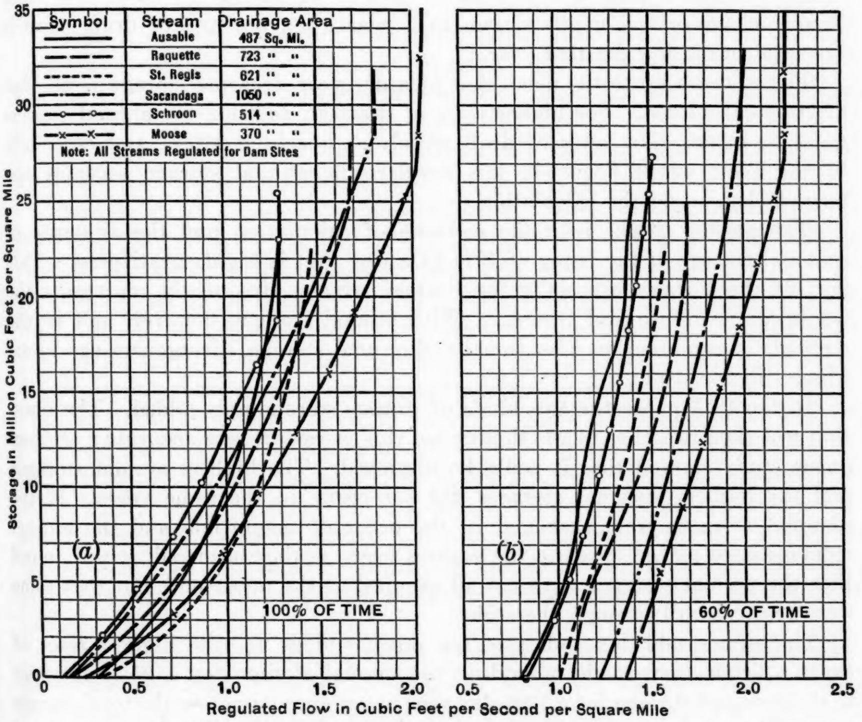


FIG. 34.—CHARACTERISTIC CURVES OF VARIOUS STREAMS, SHOWING REGULATION POSSIBLE WITH VARYING AMOUNTS OF STORAGE.

ALLEN HAZEN,† M. AM. SOC. C. E. (by letter).‡—This paper contains a contribution of real importance to the methods of making storage calculations. The use of a mass curve, in which is shown only the accumulated surplus or deficiency as compared with the mean, permits a more convenient representation and accurate study of the data. This is particularly true for high rates of draft.

The author shows the method applied to annual quantities, but it is equally applicable to monthly quantities and can be used for the whole storage calculation for the higher rates of draft. The calculations may be prepared for plotting on calculating machines rapidly and accurately and with full checking. After having tried this method on an example the writer wonders that it was not done long ago.

* *Proceedings,* Am. Soc. C. E., April, 1926, Papers and Discussions, p. 614.

† Cons. Engr., (Hazen & Whipple), New York, N. Y.

‡ Received by the Secretary, February 1, 1927.

It may be recalled that in the writer's paper entitled "Storage to Be Provided in Impounding Reservoirs for Municipal Water Supply,"* published in 1914 and subsequently herein referred to as the paper of 1914, storage was divided into two parts, namely, seasonal storage, or the storage required to hold the water of the wet part of any one year and make it available in dry months next following, and, annual storage, or the storage of water held more than one season and finally drawn in a year when the rate of draft exceeds the total mean flow for that year.

For storages up to the mean flow of a dry year, seasonal storage is all that need be considered. For higher rates of draft an addition for annual storage must be made. For rates of draft closely approaching 100%, and especially in the West where flows are less regular, the annual storage becomes the largest element in the calculation.

The author checks both the methods of calculation and the amounts of seasonal storage of the paper of 1914 in a way that is highly gratifying. This part of the storage, especially for Eastern streams, frequently represents the whole range of use and interest. With annual storage, however, and in the methods of combination of seasonal and annual storage, divergences are found which call for study.

In the 1914 paper the two kinds of storage were simply added. The question was then raised as to whether or not this procedure was accurate.† Subsequent study showed that it could be improved. The highest annual storages and the highest seasonal storages did not occur in the same years. If the seasonal storages were arranged in the order of magnitude and the annual storages were placed opposite the various years as they occurred, it was found that the annual storages were not all grouped at the upper end, but that some were scattered all through the series.

The simple addition indicated too much storage for the highest rates of draft. Trials were made of adding the annual storage for a 95% dry year to the seasonal storage for a 70% dry year to get approximately the total storage for a 95% year; and also the other way about, of adding the seasonal storage for a 95% dry year to the annual storage for the 85% dry year to produce the same result. Both gave results that came somewhat closer to the facts.

The test of the accuracy of the procedure has been that the sum of storages computed for the higher rates of draft must be equal to the sum of the corresponding actual storages for a number of streams with the longest records. The comparison of one or two streams is not enough. The longest records at hand are not long enough to smooth out the irregularities; the considerable variations between results obtained from different streams are considered to be matters of chance and the average of all is taken as the criterion.

The next method tried was called the square root method. If the two kinds of storage followed exactly the normal law of error, it would be possible to combine them into a single series by a mathematical process, namely, by adding the median values and finding the standard variation for the com-

* *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1539.

† *Loc. cit.*, pp. 1602-1603.

bined series as the square root of the sums of the squares of the standard variations of the two separate series.

As the curves indicating storage are not normal, but are more or less skewed, the calculation can only be applied to parts of those series so short that the deviations of those parts from the normal law of error are not important. This can be done by taking the ends of the curves representing the dryest years of a series, where they are approximately straight, and continuing them as straight lines across the diagram. Figures taken from these lines may be combined as previously. It was found that this method gave results agreeing for very dry years with the actual required storages computed from the long-term records of a few streams, but for less dry years there were divergences that could not be overlooked.

The reason for these divergences turned out to be that in the series representing annual storage many of the terms, frequently more than half of them as shown by the extended straight line, were negative. Actually, the negative terms do not exist. The calculation had been made as if they did exist and the result was that these theoretical negative values pulled down the results and the calculated figures were too low except at the very end for the dryest years where there are never negative values.

Afterward, an effort was made to adjust these differences by using the foregoing method to find the value for a 99% dry year and another method to fix a second point. The second point was obtained by plotting the sum of the means of the two series on the 56% line, the latter being based on an actual count of terms in comparable data for Eastern streams. These two points were then connected by a straight line on arithmetic probability paper and values for the 95% dry year, and other desired intermediate points, taken from it.

In applying this method a rather long series of simple calculations and drawings were made once for all, and the tables and curves prepared, which have since been checked with actual records of as many streams as have been found available. This method, although arbitrary, is the most satisfactory that the writer has found. Tables and curves prepared in this manner have been published.*

The writer feels that this method has been tested by practical experience to the point where, within reasonable certainty, it is in accordance with the facts and is a safe working method; but a better theoretical method of combination is to be desired.

The author's estimates for required storage for the higher rates of draft where annual storage is involved are much lower than those reached by the writer in 1914, and in his subsequent work. The difference seems to be one of definition.

The original method was to assume an indefinitely large reservoir full at the beginning of the record period; to compute the maximum depletions occurring in successive annual periods; and to make a series of these depletion values. The amount of storage required to maintain service in a 95% dry

* American Civil Engineers' Pocketbook, 4th Edition, 1920, and American Water Works Association Manual of Practice, 1925.

year was taken directly from the curve of these results. In other words, with an assumed constant draft and unlimited storage, the storage shown by the calculation is that which would be exceeded during 5 years in 100, or in whatever other ratio was desired.

The author has followed another assumption, namely, that the reservoir was of limited size (the size to be found by the subsequent procedure) and that, when it was once emptied (by failure to maintain full service), the record up to that point was closed and a new calculation was started with the reservoir empty and carried forward in the following years.

L. Standish Hall, Assoc. M. Am. Soc. C. E., made * assumptions in regard to this on a still different basis, which also produced results smaller than those reached by the writer. As a basis of statement and calculation, any assumption may be made, and it is a fair field in which each has his choice. As a matter of practical application and procedure, in order that the results may be useful and safe, it is necessary that the assumptions should be reasonably in accordance with the conditions of actual use of the structures that will be effected by the calculations.

Using the writer's method and applying it to a 95% dry year, which seems to be the best ordinary basis of rating water supplies in the United States, there is a probability, as stated in the Water Works Manual, that the full supply can be maintained for 95 out of each 100 years and that in the other 5 years, there will be shortages with deficiencies ranging from 1 to 10% (to somewhat more than 10% at long intervals) and averaging about 6% for all the years of shortage, these being 5% of the whole number.

With the assumptions made by the author it is clear that smaller reservoirs would result and the deficiencies for the years of shortage would be more than 6% and might occasionally be very much more. The author may have had power-houses in mind in making his assumption, but the writer confines his remarks to calculations for public water supplies. It may be that this difference in point of view goes to the heart of the matter.

In the writer's conception of the problem, there is no thought of stopping the draft for any interval when the reservoir is empty, or of limiting it to the natural flow of the stream for that period, or to any other greatly reduced amount. Instead it is his idea that as storage becomes depleted, consumption will be curtailed by measures known to water-works men, such curtailment not exceeding about 10% of the normal output, and that the storage provided must be sufficient to maintain such slightly reduced service throughout the whole dry period and until larger flows are again available.

The author has kindly furnished in his paper the necessary data for a comparison. Referring to total required storages on the Croton River,† and passing the higher rates of draft which are considered impracticable or inadvisable in the water-works business, it is stated that, for a draft of 0.9 of the mean flow, the computed storage by the author's method is 0.69 of the mean flow for 1 year. From the actual record for 47 years he finds that this storage would maintain the desired rate of draft for 43 years, or 90.4% of all, with deficien-

* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 249.

† *Proceedings*, Am. Soc. C. E., December, 1926, Papers and Discussions, p. 1935.

cies of 2, 5, 22, and 11% in the 4 years of shortage; that is, instead of actually maintaining the supply for all but 5% of the whole number of years, the calculated amount of storage falls short in 9.6% of the years, or twice as often as it ought to do. Moreover, by the author's calculation in one year there would be a shortage of 22% in available supply which is an intolerable assumption in public water supply.

For comparison with this, following the method of the American Civil Engineers' Pocketbook or the American Water Works Association Manual, the required storage estimated by the writer for that draft is 0.93% of 1 year's mean flow. The storage over that computed by the author, 0.93 instead of 0.69, an excess of 0.24, is sufficient to absorb the entire deficiency found by the author for the one year of greatest shortage.

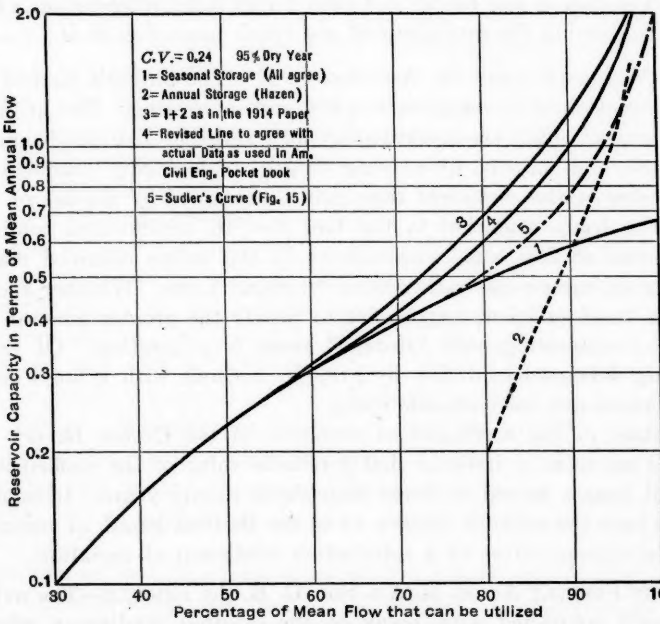


FIG. 35.—STORAGE DIAGRAM.

The matter may be shown graphically. Various storage lines have been drawn on Fig. 35. These calculations all relate to one assumed case, but the case is a representative one. Line 1 shows the seasonal storage and it is in regard to this line that there is complete agreement between the author and the writer. Line 2 shows the annual storage as computed by the writer, that is, the storage required to carry the dry years. This is over and above and does not include seasonal storage. It is separately computed by a procedure described in the paper of 1914. Line 3 is that found by the addition of numbers in Lines 1 and 2 and represents total storage as used by the writer in 1914.

Line 4 is plotted from the figures in the American Civil Engineers' Pocketbook, and results from the application of the procedure that has been briefly

outlined. It will be seen that the rating of a source by using this line will be greater for all the higher rates of draft and sometimes will be as much as 3% greater than that found by using the procedure of 1914. Line 5 is plotted from the values given by the author.

The suggestion is made to water-works men that the values for storage given in the American Civil Engineers' Pocketbook will probably be more comfortable to use in the long run.

The study of storage on a tributary is interesting. The writer has an arbitrary rule which seems to be in substantial agreement with the diagrams presented by the author. This is to the effect that storage on a tributary on Eastern streams up to about 0.9 of the mean flow is substantially as useful as a like amount of storage in a reservoir controlling the entire catchment. Additional storage is less useful and even a very large reservoir on a tributary is not useful beyond the equivalent of one year's mean flow to it.

ROGER W. ARMSTRONG,* M. AM. SOC. C. E.—The author's method of computing the coefficient of variation is novel and interesting. The utilization of "storage" years, which are intervals between times of like conditions in the run-off cycle and may be of varying length, undoubtedly produces a more accurate value of the coefficient than may be obtained by the use of calendar years. This draws attention to the fact that in arithmetical computations of deficiencies of flow some inaccuracies in the values obtained may result from using calendar years rather than "storage" years. Whether such errors would ever be of sufficient importance to justify the greater amount of labor required for computing with "storage" years is a question. Of course, in determining deficiencies in flow by graphic methods with a mass curve, the "storage" years are used automatically.

The values of the coefficient of variation of the Croton River† are significant. They seem to indicate that a reliable value of the coefficient cannot be obtained from a record of fewer than about twenty years. It would be of interest to have the author's opinion as to the shortest length of record necessary for the determination of a satisfactory coefficient of variation.

H. ALDEN FOSTER,‡ ASSOC. M. AM. SOC. C. E. (by letter).§—The writer was rather closely connected with many of the original studies on which this paper is based, and therefore has examined it with a great deal of interest. In reviewing the analysis and methods given by the author, after an interval of several years, the writer is again convinced that they are logically developed from the general assumptions on which they are based. These assumptions may be summarized as follows:

1.—The duration curve of daily flow for the 95% dry year (or any other dry year) is directly proportional to the corresponding curve for the average year, the proportional factor being the ratio of the total flow in the dry year to the total flow in the average year.

* Div. Engr., Board of Water Supply, New York, N. Y.

† *Proceedings*, Am. Soc. C. E., December, 1926, Papers and Discussions, p. 1939.

‡ With Parsons, Klapp, Brinckerhoff & Douglas, New York, N. Y.

§ Received by the Secretary, January 8, 1927.

2.—The mass curve of daily flow for the dry year is directly proportional to the corresponding curve for the average year, with the same proportional ratio as before.

3.—The year requiring the greatest seasonal storage also produces the lowest total run-off.

4.—The maximum annual storage is required in the same year in which the seasonal storage is a maximum.

It would probably be impossible to prove definitely that any one of these assumptions is true in all cases. On the other hand they undoubtedly are descriptive of general conditions in Nature. A theory based on these assumptions must be considered, therefore, as reliable in a qualitative or relative manner, but not as giving absolutely precise results in any given case. The writer believes that the tests given in the paper clearly illustrate this point.

Moreover, as the author indicates, a stream-flow record covering only a few years is certainly not long enough to show, by itself, what the true storage requirements for that particular river would be over any considerable period of years. For such a purpose, the results published by Mr. Hazen,* and used by Mr. Sudler in preparing his seasonal storage diagrams, are of great value. The method by which the author combines seasonal with annual storage is logically correct, and an improvement over previous methods.

The only point of view from which objection might be raised to the author's analysis is in connection with the development of the annual storage curves. The determination of the required annual storage is undoubtedly a very difficult problem, and one to which it may be impossible to give a universal solution. It can only be determined by a study of accumulated long-term records, or by the construction of an artificial record, such as that described by Mr. Sudler. Whether one method of deriving annual storage curves is more reliable than another would be largely a matter of opinion. The writer believes that the annual storage curves presented in the paper are entirely consistent, and as reliable as any that can be developed from the present available data.

These annual storage curves were developed from the theoretical duration curve of annual run-off described by the writer.† It was brought out in the discussion of that paper that there are other types of duration curves which are more applicable to certain streams than the curve originally presented. If a different duration curve had been used by the author, his annual storage curves would probably have been slightly different. It should be noted, however, that two duration curves having the same coefficient of variation will not differ greatly for values less than the mean run-off, up to 80 or 90% of the time. As annual storage usually is required for a group of consecutive dry years, the maximum annual storage is as likely to be required for a series of moderately dry years as for a short group including the extremely dry years. Consequently, a change in the duration curve of annual run-off which only affects the lower end of the curve but does not materially change the

* *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1539.

† "Theoretical Frequency Curves and Their Application to Engineering Problems," *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 142.

more numerous points closer to the average run-off, will not have any great effect on the annual storage requirements.

This same line of reasoning should apply to small changes in the coefficient of skew, for a change in this coefficient has a much greater effect on the extreme ends of the duration curve than on the intermediate portion. Moreover, an increase in the *CS*, while it increases the number of years with deficient flow, at the same time increases the flow in the dryest years. It may be stated, therefore, that the curves prepared by the author, which were computed for a coefficient of skew of 0.6, should be applicable to streams the *CS* of which is between 0.3 and 0.9, or possibly for an even greater range.

It is, therefore, the writer's opinion that Mr. Sudler's curves may be used for streams having a considerable range in run-off characteristics. Even for streams that are clearly outside the range of these curves the diagrams might be used for comparative purposes, where precise quantitative results are not essential.

In any case, one of the important uses of the diagram should be for the purpose of studying the relative effects of different reservoir and power-house sites on a particular stream. The results to be obtained for this purpose may be considered as entirely consistent, and as having a large degree of precision when the characteristics of the stream are within the limits specified for the diagrams.

JOEL D. JUSTIN * M. AM. SOC. C. E. (by letter).†—The methods of studying storage possibilities developed by Mr. Sudler will doubtless find considerable application where regulation for maximum dependable flow is required and where a large number of storage possibilities must be studied. In such cases the method should greatly shorten the rather laborious computations that are usually made.

However, after the reservoir and its size have once been determined, it will always be advisable, the writer believes, to follow through, by the old-fashioned methods of computation, the operation of the reservoir month by month (and even day by day during critical periods) for the period of record. Such computations show the water added to the reservoir, the water released to maintain the required flow at the given point, and the resulting elevation and contents of the reservoir. They have the advantage of showing what the reservoir would have accomplished had it been in operation during the years of record.

Whereas engineering studies and investigations are conducted to determine the truth, the resulting report is usually presented with the object of either getting some one to spend money or to prevent him from spending it. The old-fashioned methods of computation will always have, the writer believes, merit for inclusion in such reports, as they have the advantage of being easily understood by all engineers and most power company executives.

The method of building up an artificial long-term record as used by Mr. Sudler‡ by chance drawings from the years of actual record, is certainly

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† Received by the Secretary, February 18, 1927.

‡ *Proceedings*, Am. Soc. C. E., December, 1926, Papers and Discussions, pp. 1920–1921.

open to some objections. The variations in rainfall and run-off from year to year on a given water-shed are largely unexplained, but this does not mean that the sequence of wet and dry years is wholly a matter of chance Any method of extending a run-off record is open to objections. Although such records are often very short, it is usual to have rainfall and meteorological data available for a much longer term of years. By the proper use of such data, considered in connection with the existing run-off records, the writer believes that it is usually possible to build up reasonably accurate records of synthetic run-off, extending the actual records backward for many years.

Short as most run-off records are, many of them are long enough to cover marked changes in the physical character of the water-sheds. Water-sheds which, when records were started, were well forested now have had these forests replaced by brush or farms, towns, and cities. In other cases, lakes and swamps have been drained and, in others, reservoirs have been added. Such changes, however, probably affect the distribution of the run-off throughout the water-year more than they do the total amount of run-off.

The paper deals with the problem of maintaining the maximum low-water flow either at the reservoir outlet or at some point down stream where the water-shed area is much larger. With the growth of power companies and their extensive interconnection by means of long transmission lines the problem of storage from the standpoint of the engineer engaged in hydro-electric work is rapidly becoming much more complicated. The usual problem is no longer to maintain a maximum dependable flow at some given point on a stream, but rather to release the water in the reservoir so that it will produce the maximum in firm power and primary energy in a power system, which may embrace many hundred square miles of territory and extend over several States.

Such a study may embrace not only the reservoir and the hydro-electric plants on the stream in question, but also a considerable number of interconnected hydro-electric plants on other widely scattered streams, as well as existing storage reservoirs and steam plants. Such a method of use when practicable will show a much greater income creditable to the storage reservoir than by using it to produce the maximum dependable flow at some particular point on the stream. This method of use also results in over-storage; that is, the economic size of reservoir will be larger than when regulation is for maximum dependable flow. In fact, the water in the reservoir is considered as so much potential energy to be used where it will do the most good in increasing the firm power in the system. The whole stream on which the reservoir is located becomes a storage reservoir of energy, as it were. For this kind of regulation, which is becoming increasingly important, an ideal stream would be one possessing one or more economical sites for storage with a maximum amount of head capable of economic development in relatively high-head plants below the storage site and with all sites under one ownership or under sympathetic ownership.

A relatively simple case will be used to demonstrate this method of regulation. In Fig. 36, River X has a large storage reservoir; also located on it are Plants A, B, and C, which are high-head plants with relatively short

penstocks, (that is, low cost per kilowatt of installed capacity). The storage reservoir is larger than would be required to maintain maximum dependable flow at, say, Plant B; and Plants A, B, and C have installations much greater than would be justifiable if River X were the company's only source of power.

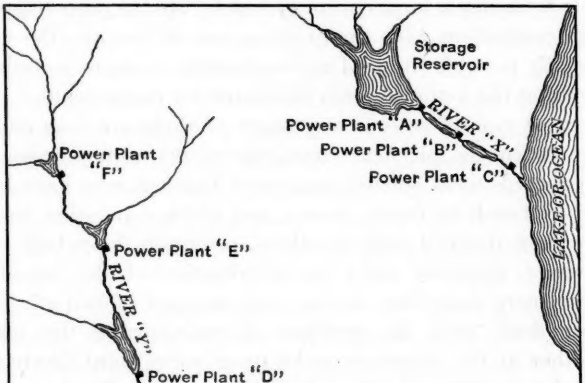

Fig. 36.

River Y is a much larger stream the water-shed of which may be at a long distance from River X. This river is uncontrolled, has a relatively flat slope, and is subject to wide fluctuations in flow. Plants D, E, and F are controlled by the same company which controls Plants A, B, and C, on River X. The outputs of all the plants feed into the same system.

During periods of minimum flow on River Y a maximum quantity of water is released from the storage reservoir on River X, and Plants A, B, and C are working at full capacity during the peak hours. Perhaps, during 50% of the time over a term of years, some water will be released from the storage reservoir on River X, and during the remainder of the time there is very little energy produced at Plants A, B, and C on River X.

On River Y Plants D, E, and F have their low-water output augmented by Plants A, B, and C, on River X. During the remainder of the time the plants on River Y carry the load (except for short-time peaks which may be carried by Plants A, B, and C).

In some cases, plants of the character of A, B, and C which operate only 10% of the time are found to be economically justifiable. Of course, every practical case has many more complications but these suppositions serve to illustrate the principle of the operation of storage reservoirs for the maximum production of primary energy in a system.

Steam plants often enter into storage studies to a surprising extent. In one case the writer found that it was economical to draw down a certain reservoir every two or three years and on those occasions put into service for a brief period of time an antiquated steam plant which would otherwise be junked. Possibly, there are many cases where this procedure might pay, as it makes the reservoir water do more work and leaves less dead

storage in the reservoir. Furthermore, every large power company has several old steam plants about ready to be junked, but which could be kept in condition ready to operate at, say, a month's notice without too great an annual stand-by cost.

L. STANDISH HALL,* ASSOC. M. AM. SOC. C. E. (by letter).†—The author is to be congratulated for his timely paper on this important subject. In a recent discussion‡ of the paper§ by R. D. Goodrich, M. Am. Soc. C. E., the writer called attention to the fact that the recent developments in the treatment of skew frequency curves would permit of the improvement in the results obtained in applying probability methods to the study of storage requirements.

One of the principal difficulties encountered in determining storage requirements, is the proper combination of seasonal and annual storage. If the draft to be used is less than the minimum annual flow of the stream, the study is greatly simplified, as in this case only seasonal storage need be considered. For the purpose of determining the amount of such storage, the curves developed by Allen Hazen, M. Am. Soc. C. E., appear to be satisfactory.‖

The new term introduced in the paper, "monthly storage," appears to be unnecessary. In Fig. 1,¶ showing a typical mass curve, the author has considered the years to be divided by the peaks. Using this division of the year, the monthly storage term becomes necessary in computing the increment to be added to the annual storage. However, such a division of the year (called by the author the storage year) does not coincide with the normal water-year. The water-year begins either in the fall, or else coincides with the calendar year, and includes all the run-off occurring in one annual cycle. An annual cycle should logically include the run-off occurring between periods of low summer flow.

Considering the mass curve in Fig. 1, the storage is drawn down during the first year by the seasonal storage indicated by the ordinate, a. In the second year the reservoir did not refill, and at the end of that year the depletion of the reservoir at b is the total ordinate between the draft line and the mass curve. The amount of annual storage is this ordinate minus a. Similarly, in the third year, the annual storage is the ordinate c plus e minus a. In other words, the amount of seasonal storage to be considered in any period of depletion of a reservoir, extending over two years or more, is the amount occurring in the first year. It is evident that the seasonal storage, a, is independent of the succeeding years and must be caused by a year having a run-off greater than the rate of draft. As any such year might begin a period of drought, a logical method of allowing for seasonal storage would be to add to the annual storages, the average seasonal storage required in years having flows above the rate of yearly draft. It is possible that further study of the probabilities

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† Received by the Secretary, February 25, 1927.

‡ *Proceedings*, Am. Soc. C. E., February, 1927, Papers and Discussions, p. 273.

§ "Straight Line Plotting of Skew Frequency Data," *Proceedings*, Am. Soc. C. E., August, 1926, Papers and Discussions, p. 1063.

‖ "Storage to Be Provided in Impounding Reservoirs for Municipal Water Supply," *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1539.

¶ *Proceedings*, Am. Soc. C. E., December, 1926, Papers and Discussions, p. 1920.

involved in the combination of the two quantities may disclose some better way of combining annual and seasonal storage.

One weakness of the author's method is that, with high rates of draft and depletion of the reservoir extending over many years, the maximum depletion may not be coincident with the year of minimum run-off. This is the assumption that the author makes in his method. In Table 1,* the maximum depletion occurs in the sixth and the tenth years, the sixth year being the year of minimum run-off. In this table, a very slight change in the run-off percentages would cause the maximum depletion to fall in the tenth year alone, and this year is only slightly less than the rate of draft. Such a condition frequently occurs with the higher rates of draft.

H. Alden Foster, Assoc. M. Am. Soc. C. E., has shown† that with a frequency distribution of a given type under the Pearsonian classification (that is, Type I, Type III, etc.) having a certain *c s*, the variations from the mean are proportional to the *c v*. The author has evidently selected the Type III curve as representing the frequency distribution to be applied to stream-flow records, since he states that the *c v* must be less than one-half the *c s*.‡ The writer has shown§ that the majority of streams having long records fall in the Type I classification. However, Mr. Foster believes that with longer records more streams would fall in the Type I class.‖

The writer recently combined the records of eighteen streams having a *c s* of approximately 0.50, by reducing the records to a *c v* of 0.10. The resulting series contained 608 terms. It was found that the frequency curve formed by this series lay between the Type III curve and the modified Type I curve, the curve still belonging in the Type I class. The Type I curves have a greater percentage of terms below the mean than the Type III curves, although the minimum values are lower in the case of the Type III curves. It may be that for the purpose of calculating storage requirements, it is immaterial which type of curve is used.

For a complete analysis of the subject, after the selection of the proper type or types of curve to be used, computations of annual storage should be made for several values of *c s* covering the range of this coefficient ordinarily encountered in stream-flow records. If the annual storage is computed for various values of *c s* and a uniform *c v*, the storage for a given *c v* and *c s* can be easily determined. The author's calculations have been made for a *c s* of 0.60 and will apply to streams falling in this classification. His statement that the curves will apply for *c v*'s of 0.10 to 0.30, is not necessarily true, since a stream having a *c v* of 0.10 might have a *c s* of only 0.20 and still fall in the Type III class.

The author's method of preparing an artificial record is open to criticism. In making the drawings, each card should be returned to the pack before a subsequent drawing is made. By the author's method a correlation exists

* *Proceedings*, Am. Soc. C. E., December, 1926, Papers and Discussions, p. 1922.

† "Theoretical Frequency Curves and Their Application to Engineering Problems," *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 142.

‡ *Proceedings*, Am. Soc. C. E., December, 1926, Papers and Discussions, p. 1924.

§ *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 187.

‖ *Loc. cit.*, p. 201.

between each drawing; in other words, after the first drawing the "universe" from which drawings are made is altered.* The mean for each 50-year period is also equal to the mean of the "universe", whereas under the normal method of drawing this would not be likely.

In plotting the annual storage, the author has used the depletions of the reservoir at the end of each year. The writer feels that this method gives undue weight to periods of maximum depletion of the reservoir, and that only the maximum depletions of the reservoir during each period of draw-down and refilling should be considered. The writer has discussed this subject rather completely in a previous paper† and will not repeat it here. He has altered his views in some respect during the meantime, but in the main, the procedure there outlined is still applicable. If an artificial record is prepared in the proper manner, as explained, and depletions in storage are computed, it will be found that, for the higher rates of draft, the storage required in the 95% year will occur only during cycles of drought at periods less frequent than once in 100 years.

Summing up the writer's views, there are two points which require especial consideration: First, as to the proper type of curve to apply to stream-flow records—it may be that further study of long records may show that stream flow can be represented by one type of curve, and that the apparent variations from this type are of an accidental nature due to the shortness of the records.‡ This would necessitate a correlation between the $c\ v$ and $c\ s$. On the other hand, it may be found that no material difference in storage requirements would be occasioned by the use of either the Type I or the Type III curve; second, further investigation is needed as to the best method of combining annual and seasonal storage.

EDWARD M. CRAIG, JR.,§ ASSOC. M. AM. SOC. C. E. (by letter).‖—The mean law of error or the probability of variation from the true, or normal, or average, is not applicable to the probable yield of water-sheds as it is, for example, to the probable error in surveying measurements in at least one important respect. The point of difference is that the number of dry years is usually greater than the number of wet years as measured by variation from the average; and the yield of the wettest year is well known to exceed the average yield by more than the amount the dryest year is deficient.

Sir Alexander Binnie, the English authority, states that from a study of 7 000 rainfall records over the world the annual rainfall is above the mean in 45.8% of the years and below the mean in 54.2% of the years, the average of the wet years being 119% and of the dry years 83 per cent. The yield of the Croton water-shed in 58 years has been above the mean in 44.8% of the years and below the mean in 55.2% of the years. During 23 years of records for the Catskill and Schoharie water-sheds, the same results are true, 52.2% of the years being wetter and 47.8% dryer than the average. The greatest

* "Introduction to the Theory of Statistics," by G. Udny Yule, pp. 335–336.

† *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), pp. 249–255.

‡ *Proceedings*, Am. Soc. C. E., January, 1927, Papers and Discussions, pp. 61–64.

§ Asst. Engr., Bureau of Water Supply, Dept. of Water Supply, Gas and Electricity, New York, N. Y.

‖ Received by the Secretary, March 4, 1927.

yearly yield of the combined Catskill and Schoharie Systems has been 150%, while the lowest yield has been 80% of the average. For the Croton, the maximum and minimum yearly yield has been 152% and 52%, respectively, of the average yield.

Where the data are sufficient, it is more accurate to use two coefficients of variation, one for the wet years and one for the dry. The coefficient for the wet years will be larger and will indicate a greater variation from the mean in a smaller number of years, while that for the dry years will be smaller and will give less variation from the mean in a greater number of years, which is in accordance with the facts.

It is more doubtful whether the period of maximum droughts can be provided for strictly according to the probability curve. There is a tendency for dry years to succeed dry years and the critical drought that must be provided against is usually found to be two, three, or four years, or even longer periods. When it is possible it is much safer to consider the droughts that have actually occurred and that may apply to a given region. Burgess* has tabulated the periods of lowest yield that have been known to occur on streams in Northeastern United States. This has advantages on the side of safety over merely using the mass curve for a single stream. It has the disadvantage that it may not take into sufficient account peculiar and local climatological conditions.

Unless the storage ratio is very high, the application of the theory of probabilities emphasizes the lack of absolute security of the so-called safe yield, when it approaches the average yield. There is a big difference between the 100% safe yield and the 95% safe yield. This emphasizes the usefulness and even the necessity of an auxiliary source of water supply, such as wells, the temporary filtration of an impure source, or a connection with the supply of another city, for which it is apparent the municipality could afford to pay ten or twenty times as much as for the water used regularly when it is necessary to make good a deficiency.

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* *Engineering News-Record,* 1922, Vol. 89, p. 970.

THE STRESSES IN A FREE PRISMATIC ROD UNDER A SINGLE FORCE NORMAL TO ITS AXIS

Discussion*

By William R. Osgood, Assoc. M. Am. Soc. C. E.

William R. Osgood,† Assoc. M. Am. Soc. C. E. (by letter).‡—This is an interesting paper. The location of the center of percussion (derivation of Equation (10)§) seems unnecessary, since it is commonly given in elementary textbooks on mechanics.

A simpler derivation of the equations of the shearing forces (Equations (13)‖ and (15))‖ and of the equations of the bending moments (Equations (14)‖ and (16))‖ occurs to the writer. According to d'Alembert's principle, the problem may be reduced to one of equilibrium if the rod is acted on by the force, P, and if at the same time each particle of the rod is acted on by a force equal and opposite to the product of the mass of the particle and its acceleration.

With the positive direction downward, the equation of motion applying in the present case gives for the rod (see Fig. 8), $-P = K l a_g$; also, with counterclockwise rotation positive, $a_g = r \alpha$, in which, a_g is the acceleration of the center of mass of the rod and α is the angular acceleration of the rod. As the acceleration of any point of the axis of the rod is proportional to the distance of the point from O, the free axis of rotation, the acceleration, a, of any point of the axis a distance, y, to the left of P, or z to the right of P, becomes,

$$a = \frac{r + x \begin{cases} +y \\ -z \end{cases}}{r} a_g$$

$$= -\frac{P}{K\,l\,r}\left(r + x \begin{cases} +y \\ -z \end{cases}\right) \qquad (27)$$

also,

$$\alpha = -\frac{P}{K\,l\,r} \qquad (28)$$

The equal and opposite forces which, according to d'Alembert's principle, are to act on the particles of an elementary slice of the rod taken between

* This discussion (of the paper by Joseph N. Le Conte, Esq., published in January, 1927, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

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‡ Received by the Secretary, January 13, 1927.

§ *Proceedings*, Am. Soc. C. E., January, 1927, Papers and Discussions, p. 6.

‖ *Loc. cit.*, p. 7.

two adjacent cross-sections, may be replaced by the force, $-\,a\,dm = -\,a\,K\,d\,l$, and the couple, $-\,\alpha\,d\,I = -\,\alpha\,k_0^2\,K\,d\,l$. The loads to be applied to the rod are, therefore, those shown in Fig. 8, namely, the force, P; a uniformly varying load, $-\,a\,K$, per unit of length; and a uniform distribution of couples, $-\,\alpha\,k_0^2\,K$, per unit of length.

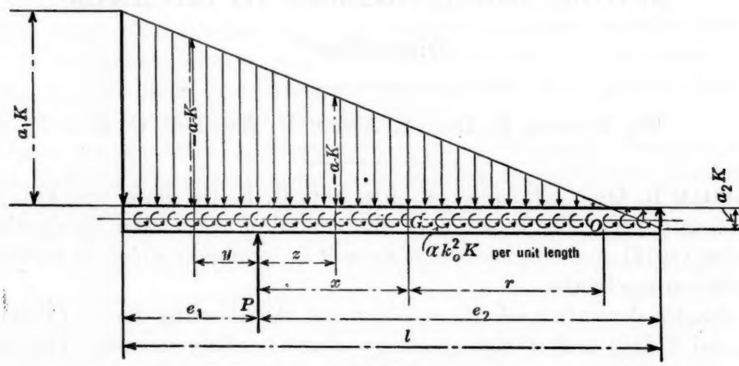

FIG. 8.

The shearing force at any section a distance, y, to the left of P is, with the usual convention for positive and negative shears,

$$Q_1 = -\,\frac{(-\,a_1 - a)\,K}{2}\,(e_1 - y)$$

in which, a_1 is the value of a at the left end of the rod, that is, the value for $y = e_1$. By substituting a and a_1 obtained from Equation (27) and reducing, Q_1 becomes,

$$Q_1 = -\,\frac{P}{l\,r}\,(e_1 - y)\left(r + x + \frac{e_1 + y}{2}\right)$$

The bending moment at the same section is, with the usual convention for positive and negative moments,

$$H_1 = -\,\frac{-\,a\,K\,(e_1 - y)^2}{2} - (-\,a_1 + a)\,K\,\frac{e_1 - y}{2} \times \frac{2\,(e_1 - y)}{3} + \alpha\,k_0^2\,K\,(e_1 - y)$$

By substituting a and a_1 obtained from Equation (27) and α from Equation (28) and reducing, H_1 becomes,

$$H_1 = -\,\frac{P}{2\,l\,r}\,(e_1 - y)^2\left(r + x + \frac{2\,e_1 + y}{3} + \frac{2\,k_0^2}{e_1 - y}\right)$$

The shearing force at any section a distance, z, to the right of P is,

$$Q_2 = -\,\frac{(a_2 + a)\,K}{2}\,(e_2 - z)$$

in which, a_2 is the value of a at the right end of the rod, that is, the value for $z = e_2$. By substituting a and a_2 obtained from Equation (27) and reducing, Q_2 becomes,

$$Q_2 = \frac{P}{l\,r}\,(e_2 - z)\left(r + x - \frac{e_2 + z}{2}\right)$$

The bending moment at the same section is,

$$H_2 = \frac{a\,K\,(e_2 - z)^2}{2} + (a_2 - a)\,K\,\frac{e_2 - z}{2} \times \frac{2\,(e_2 - z)}{3} - \alpha\,k_0^2\,K\,(e_2 - z)$$

By substituting a and a_2 obtained from Equation (27) and α from Equation (28) and reducing, H_2 becomes,

$$H_2 = -\frac{P}{2\,l\,r}\,(e_2 - z)^2\left(r + x - \frac{2\,e_2 + z}{3} - \frac{2\,k_0^2}{e_2 - z}\right)$$

By substituting for r in the denominator of the right-hand side of the equations for Q_1, H_1, Q_2, and H_2, the value given by the author's Equation (11),* the equations just derived for Q_1 and Q_2 become the same as the author's Equations (13) and (15), except that the sign of Q_1 will be opposite, and the equations here given for H_1 and H_2 obtain a form to which the author's Equations (14) and (16) can easily be reduced with opposite signs. The opposite signs in the three cases mentioned are due to the fact that the usual convention for the signs of shearing forces and bending moments has been followed throughout in this discussion.

The following brief bibliography may be of interest in connection with this paper:

Zschetzsche. *Zeitschrift* des Vereines deutscher Ingenieure, 1894, p. 134.
Flamant, A. Resistance des Materiaux. Paris, 1897, p. 635.
Ostenfeld, A. Teknisk Elasticitetslaere. Copenhagen, 1924, p. 293.

* *Proceedings*, Am. Soc. C. E., January, 1927, Papers and Discussions, p. 6.

ECONOMIC AND ENGINEERING PROBLEMS OF HIGHWAY LOCATION

Discussion*

By Messrs. Fred Lavis and George E. Goodwin

Fred Lavis,† M. Am. Soc. C. E.—The application of any economic theory of highway location is even more difficult and complicated than is the case in railway location, on which latter the former must necessarily be based.

The problem of handling the traffic that passes back and forth, to and from New York City and the South and West, through the Metropolitan Area of Northern New Jersey, has long been recognized as one of major importance. There is not only the local traffic between New York City and the manufacturing and residential districts of the Area, but also the through traffic to and from the West *via* the State of Pennsylvania, the traffic to and from Philadelphia, Pa., Baltimore, Md., Washington, D. C., and the South, and the traffic to and from the New Jersey coast resorts.

The rate of increase in this traffic is shown by the following traffic counts made at Rahway, N. J., on the Lincoln Highway. They show a 24-hour count on a day in July for each year:

1921	12 153	1924	26 573
1922	16 610	1925	No count
1923	18 314	1926	43 735

Other counts show similar increases on other routes.

Up to the present the traffic across the Hudson River has been handled on the various ferries and more or less distributed, but, in 1927, when the Holland Vehicular Tunnels under the river are completed and in operation a large part of this traffic will be concentrated at one point in Jersey City, N. J. The congestion in Newark, N. J., is already so bad that not only is the through traffic delayed, but local business is greatly hampered.

To meet this situation the New Jersey State Highway Commission has planned and is building an entirely new highway from the New Jersey end of the Holland Tunnels through Jersey City and Newark to the westerly side of Elizabeth, N. J. The length of this highway will be about 13 miles and its cost will exceed $30 000 000.

It will have a roadway 50 ft. in width; there will be no curves of less than 1 000-ft. radius, and the gradients will not exceed 3.5 per cent. There will be no grade crossings with other highways and, of course, none with railways.

* Discussion on the paper by W. W. Crosby, M. Am. Soc. C. E., continued from February, 1927, *Proceedings*.

† Cons. Engr., Jersey City, N. J.

Connection will be made at suitable intervals with the most important streets and highways which this new route crosses, by means of ramps, entering and leaving in the direction of the traffic.

The 50-ft. width of paved roadway will provide for five lanes of traffic, two in each direction, with one spare for emergencies, and it is expected that the traffic will be of a volume approaching 20 000 000 vehicles annually.

It will be realized, therefore, that the question of location and design is one of prime importance in a project of this nature, having more of the characteristics of a four-track main trunk railway through a densely built-up region than of an ordinary street or highway.

The route, of course, must be laid out primarily to serve the traffic, to take the people who desire to use it as closely as may be to and from the places between which they desire to travel. It must be laid out with a due regard for the cost of acquiring the necessary right of way and to avoid unnecessary length, rise and fall, and curvature. The costs of construction and of right of way must be balanced against the increase of operating costs due to increase of length, rise and fall, or curvature. In designing this route it was found that data in regard to these latter items were very scarce, but studies were made and formulas developed to determine the differences in operating costs.

As in railway location, any theories of this kind must be developed and used with great care and judgment—mere mechanical mathematics are worse than nothing; but, as in all branches of engineering, the mathematical formulas should be developed and used as an aid to good judgment.

George E. Goodwin,* M. Am. Soc. C. E. (by letter).†—To one who has given much time, and even more thought, to highway location, this paper appeals as presenting, in a broad way, the principal highway location problems. Necessarily, not all highway projects are affected by many of the problems listed; also, some projects might have certain important affecting conditions not mentioned in the paper. This would be especially true of scenic mountain highways in the Western United States.

The principal difficulties in the way of correct highway location lie in the present methods of considering and determining the project in question solely in relation to itself and the immediate effect that it will have on the communities served. Frequently immediate benefits would be as great and ultimate benefits much larger, and more lasting and far reaching, if the project was studied and outlined in relationship to the other roads of that section and of the State system, and even in some cases of the National system, of highways; not only to the highways that are now improved or about to be improved, but also to the logical routes for important through highways that are reasonably sure to be built in the not distant future. Such considerations of a project can prevail only when the matter has been removed from the realm of politics and local community greed and jealousy.

Obviously it is impossible for elective or appointive laymen to consider fully and decide wisely all the various economic and engineering problems of

* Cons. Engr., Hood River, Ore.

† Received by the Secretary, February 28, 1927.

highway location and improvement, and as long as the engineering expert's studies and activities are directed and his findings approved or disapproved by a commission of political appointees with the ideas of whom the engineer must be in harmony, the results are bound to be mediocre. There are, of course, some wonderful exceptions to these conditions. Some State executives and commissioners have been broad and visioned men, considering the highway system from the viewpoint of the State at large, and have insisted that the highway systems improved be economical for the whole State in both engineering and commercial aspects. As a consequence in some States the highway systems have been developed along economical and logical lines. Public education on this subject is bound to bring beneficial results.

While all the physical conditions and standards as to length, alignment, etc., presented in the paper are important, the writer agrees that altogether too much stress has been placed in the past on limitation of gradient. What might be practical and necessary as a standard of limitation for alignment or gradient under certain conditions of topography, traffic, etc., would be absolutely wasteful and unnecessary under other widely dissimilar conditions. In many States more miles of equally useful, and oftentimes more pleasing, highways can be had if greater latitude is allowed in these matters both by the State Highway Departments and by the Federal Government. Statistics of motor accidents show that more accidents have occurred on relatively straight and level reaches of roads than on those having relatively steep grades and sharp curvature. Although, of course, neither steep grades nor sharp curves are desirable and should be avoided wherever practicable, it is a waste of public money and poor engineering to insist that all classes of roads in the system of a mountainous State should have, say, a maximum gradient of 5% and a minimum radius of 300 ft. for curves. Likewise, under certain conditions, standard widths of 24 ft., or more, are impractical.

As Mr. Crosby has brought out, all factors must be fully considered for an economic solution of these problems. In highway location, as in any other improvement work, true engineering economy consists in securing that which is best and obtainable for the funds, for the time being, and for the future as it may be visioned; and, while in some cases this result may be attained, it is probable that even the greatest successes of the present will afford ample reason for criticism by the engineer of fifty years hence, who will see only the results in the light of that time but be unacquainted with the limitations and requirements that guided the original builder.

NORTH CAROLINA BITUMINOUS EARTH ROADS

Discussion*

By Carlton N. Conner, Assoc. M. Am. Soc. C. E.

Carlton N. Conner,† Assoc. M. Am. Soc. C. E. (by letter).‡—The author all too briefly describes one of the rapidly developing and improved methods of increasing the traffic capacity of a low type road at a moderate cost.

There are in the United States more than 368 000 miles of surfaced rural highways which in their present condition are suitable for only about 500 or 700 vehicles per day. This is nearly 80% of the total surfaced mileage; the remaining 20% are surface treated or surfaced with many types of treatments and paving up to and including Portland cement concrete pavements. The total mileage of the country is estimated at more than 3 000 000 of which approximately 85% has no surfacing; 10% has a low type of surfacing, such as the sand clay, top soil, etc., mentioned by Mr. Catchings; 4% has an intermediate type of surfacing ranging from surface treatments up to and including bituminous concrete on stone base; and less than 2% has a high type of surfacing, such as Portland cement concrete base with bituminous, brick, or block top course, and plain and reinforced concrete pavements. The bituminous earth roads of North Carolina are a step in progressive or stage construction which changes the low types of surfacing consisting of sand clay, top soil, etc., to the intermediate type.

A highway surface may be considered as low type, intermediate type, or high type, depending primarily on its traffic capacity, and secondarily on its first cost, maintenance cost, and salvage value. The type used in North Carolina has been classed in other States as a skin surface treatment and would fall into the class designated as a medium or intermediate type.

There are of course several kinds of surfacing which may be properly classed as intermediate types, such as the mixed-in-place surfaces, sand asphalt, veneer macadams, and others which to a large extent utilize local materials for their base course and local or shipped-in materials for the wearing or top course. A majority of the intermediate type surfaces have bitumens as binders, tar is frequently used as a prime coat and asphalt for the second and subsequent treatments. In some States tars are used almost entirely, whereas others are inclined to use asphalts. Freight rates and deliveries are sometimes deciding factors in the selection of the class of bitumen.

* This discussion (of the paper by William B. Catchings, Assoc. M. Am. Soc. C. E., presented at the meeting of the Highway Division, New York, N. Y., January 21, 1926, and published in January, 1927, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Chairman, Investigation of Low Cost of Improved Roads, Highway Research Board, National Research Council, Washington, D. C.

‡ Received by the Secretary, February 8, 1927.

In New England this type of surface treatment extends over about 2 000 miles of road using the local gravel as aggregate and tar as a binder. In Long Island and New Jersey are found hundreds of miles of road having the local sand loam mixed with asphaltic oils. In South Carolina, Virginia, Tennessee and Pennsylvania, as well as some of the Middle and far Western States similar medium-type surfacings are being investigated, used, and developed. The actual materials vary with local conditions, but the objects are similar in nearly all cases—to improve the low type surfaces by the addition of a low cost wearing surface.

These new surfaces, such as those developed in North Carolina, will increase the traffic capacities from 500 or 700 vehicles per day to 1 000 or 2 000. There are instances, such as in Long Island, where a daily traffic capacity of more than 5 000 cars and light trucks is claimed. It is not to be presumed that these surfaces will ever take the place of, or even compete in traffic capacity with, the high types of paving, but they will undoubtedly fill in many a gap and missing link of highway between objectives on trunk lines as well as form an important part in the construction of feeder roads, of development roads, and of roads that carry a large seasonal tourist traffic.

RECENT DEVELOPMENTS IN CONCRETE PAVEMENTS

Discussion*

By Messrs. Elmer G. Hooper, T. M. Ripley, W. C. Hammatt, R. W. Crum, and Carlton N. Conner.

Elmer G. Hooper,† Assoc. M. Am. Soc. C. E.—The question of the triangular slab pavement has been raised. The rigid-slab type of pavement as usually laid on a natural soil sub-grade is not susceptible of exact analysis for design. It is an indeterminate structure and the important unknown factor is the support. The condition and character of the supporting soil in many cases may influence the amount and distribution of stresses in the pavement only slightly, but a poor sub-grade presents a serious problem. The structural stresses in the pavement are reversible and become excessively large for economical design. If a pavement can be built in which the units rest on permanent supports at known points, it provides a structure the behavior of which can be predicted from analysis or experiment. Any type of supported slab pavement that offers simplicity in design, reasonable ease of construction, and moderate cost is preferable to the indeterminate slab on a poor base and should find an extensive field of use.

A pavement made up of triangularly shaped slabs supported at their corners was conceived as satisfying the conditions enumerated. The three-point support of units assures definite stresses with no reversal even if the supports settle unevenly. The shape and economical size admit of precasting, transporting, and assembling the units in the pavement expeditiously. It is possible to remove slabs from the pavement for replacement or salvage.

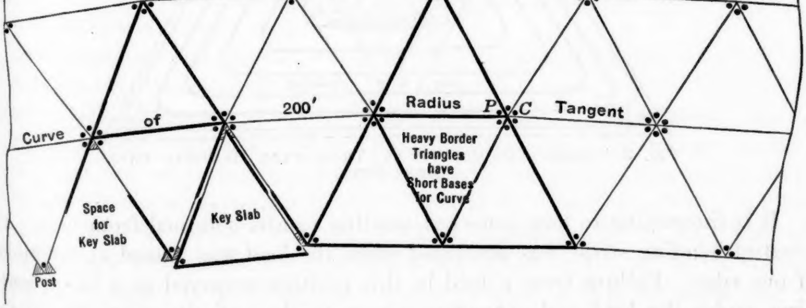

Fig. 1.—Triangle Highway Pavement, Assembly of Slabs.

The triangular units are assembled as shown by the plan for a two-lane road in Fig. 1. Each lane is made up of a row of triangles placed edge to

* Discussion on the paper by H. Eltinge Breed, M. Am. Soc. C. E., continued from February, 1927, *Proceedings*.

† Asst. Prof., Civ. Eng., New York Univ., New York, N. Y.

edge, their altitudes representing the lane width and the bases forming alternately the left and the right side of that lane. In this manner three slabs come together for support on a single pile or post. Where two lanes are contiguous the supporting posts under the line of contact carry the corners of six slabs each. Contiguous edges of slabs are provided with special interlocking lugs and recesses so that deflections and stresses in one may be shared by its neighbors. Loosely fitting pins through the vertices of the triangles into the posts prevent undue lateral movement after assembly.

The triangular slab is as nearly determinate as any plane surface or slab. Loads in any position are bound to come on or within the three lines joining the supports so that the deflection is convex downward, producing tension in the bottom and compression in the top of the slab. The direction and magnitude of these stresses can be determined because service conditions can be duplicated for test purposes. Studies already made have shown the behavior of steel plate and of reinforced concrete models of one-third scale. The results make it possible to proportion and place the steel to take the tension in the triangles. Fig. 2 shows an amount and arrangement of steel in a 7-in. slab for a pavement intended to carry loads up to 12 tons on one axle.

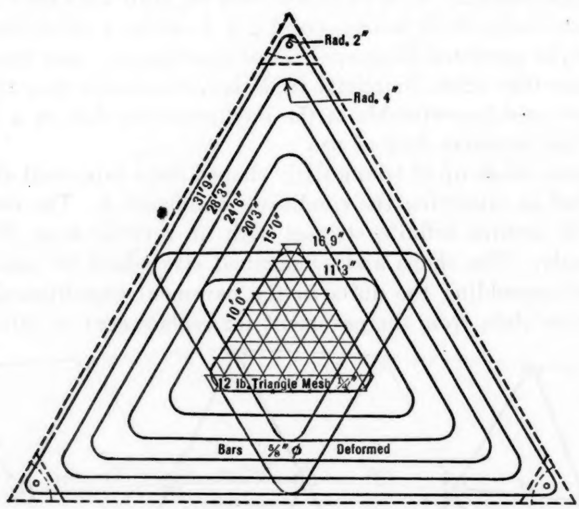

FIG. 2.—STEEL ASSEMBLY PLAN, TRIANGULAR HIGHWAY PAVEMENT UNIT.

It is interesting to note some outstanding results obtained from tests. The greatest bending stress was developed when the load was placed at the middle of one edge. Failure from a load in this position occurred as a break beginning under the load and extending across to the opposite apex, Fig. 3 (*a*). The slab can be conceived as made up of a number of strips, or beams, of unit width, stuck together and shaped like a triangle by making an outside strip serve as the base and sawing diagonally across the grain for the other sides. With this conception the slab failure mentioned corresponds to the breaking of the whole series of beams at their centers, Fig. 3 (*b*). The tests showed

conclusively that the whole cross-section represented by the altitude of the triangle was bearing the load, and the deflection of each of the unit strips, considered as a simple beam, was an index of its part of the whole load. The average load per unit strip is the total load divided by the altitude of the triangle. The deflections measured across the altitude show that the strips do not carry equal parts, instead there is a maximum at the load reducing uniformly to zero at the apex. Directly under a load on the edge the deflection corresponds to a simple beam deflection under twice the average unit load. For example, a triangle with base length of 12 in. and an altitude of 10 in., and bearing a load of 1 000 lb., would produce an average load of 100 lb. per in. of altitude, whereas the deflection under the middle of the 12-in. base would be the same as that produced in a simple beam by 200 lb.

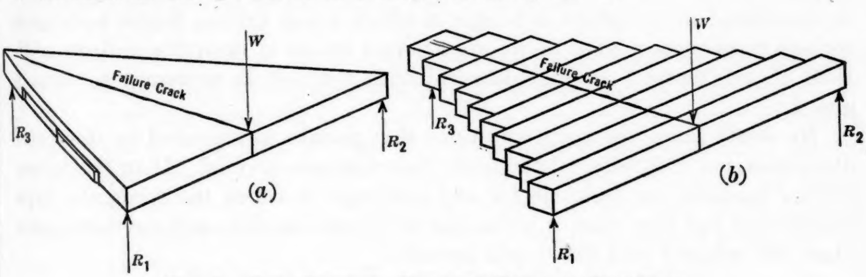

FIG. 3.—TEST RESULTS, TRIANGLE HIGHWAY PAVEMENT.

Shearing stresses are naturally most important at the apices of the slabs. The critical cross-section to resist shear depends on how far the support extends under the corner, the larger bearing giving the greater shear section and, accordingly, the lesser unit shearing stresses. In order to increase the bearing and improve contact between the slab and its support, it is proposed to use cushion blocks of treated wood, that shall extend their own thickness beyond the edges of the posts. The necessary shear-resisting area may be obtained in this way to permit the use of an allowable unit stress suited to the type of reinforcement without stirrups.

The triangular slab resting on three supports placed under its apices has advantages over slabs supported at four or more points. In the case of the latter a small settlement of one support would result in a "teetering" slab in which the stresses would become complex and excessive. The triangular slabs would show the settlement as an unevenness of the pavement without developing any appreciable change in stress distribution, and the low point can be raised and "shimmed" when it is found convenient to do so. The triangular shape lends itself more readily to handling as precast units because interlocking edges, (Fig. 3 (*a*)), can be constructed so that the units may be assembled in the pavement as a whole or any unit may be removed and replaced by the simple expedient of removing the pins that keep the corners from sliding off the posts and then moving the slab laterally about 6 in. to free it from its lock with adjoining slabs (Fig. 1).

Expansion must be provided for in this type of construction as in any other. The joints are far more frequent than in the usual type, therefore

the space to be allowed at each one is correspondingly smaller. Slabs of normal size provide two diagonal joints every 12 ft., an average of one in 6 ft. Expansion for a range of temperature of 150° is about $\frac{1}{16}$ in. in the 6 ft. A $\frac{1}{2}$-in. strip of treated felt stuck to one of the faces of a joint before the contiguous slab is pushed into contact, should provide sufficient "go and come". In case this allowance is not sufficient the angle of contact, together with the slight play at the loose pins, will permit slabs to accommodate themselves by lateral movement.

There are normally two conditions for design. In the first case the subgrade is just free of the under side of the pavement, so that failure of a unit can cause no damage to traffic and a low factor of safety can be used in the design. In the second case the pavement is far enough above the ground to be considered as a viaduct or bridge in which a slab failure might have more serious consequences and, therefore, a larger factor is desirable and, in addition, a curb about 1 ft. high integral with the slab is necessary to form a guard.

No doubt there are many questions that remain unanswered in this brief discussion, but it is believed the most important are covered. If in the future heavier loadings are desirable for any highways in which the triangular type of pavement has been used, it is feasible to salvage the slabs and use them again where the original load limits still prevail.

T. M. RIPLEY,* M. AM. SOC. C. E.—Mr. Breed has mentioned the advisability of constructing a road on piles.† There are in New York State soils of almost every known character from muck and swamp to granite mountains, yet in none of these is it possible to visualize a road on piles unless it becomes a bridge.

Unless Mr. Breed is misunderstood, this pavement on piles can be built as cheap, or more cheaply, than some other kinds. Where it can be so built it must be marsh, swamp, or open water, and it seems that in any of these locations such a structure would take more the attributes of a bridge than of a highway as the terms are commonly used.

Although the writer has had considerable experience in building and maintaining roads for the State of New York, nevertheless the road on piles is difficult to visualize and more definite information is necessary from the author in order to understand where he considers it would be economical to build such a pavement.

W. C. HAMMATT,‡ M. AM. SOC. C. E. (by letter).§—In this able paper, the author has outlined a few of the recent developments in concrete pavements, and has given some of the principles which will govern further development of this type of roadway. It could not be expected that the entire field would be covered in so short a paper, and the writer hopes that a full discussion will more nearly cover all phases of concrete pavement construction.

* Chf. Engr., Greater Motorway System of Erie County, Buffalo, N. Y.

† *Proceedings*, Am. Soc. C. E., January, 1927, Papers and Discussions, p. 35.

‡ Cons. Engr., San Francisco, Calif.

§ Received by the Secretary, January 13, 1927.

One phase of concrete slab construction has received little attention in the past. The proper proportioning of concrete mixes and the proper mix to produce the greatest strength are determined, the necessary tests being made under laboratory conditions to confirm the assumptions. This practice is now well established, but the reproduction of the results under field conditions is not so far advanced.

A comparatively recent development in California along these lines may be cited. Over large areas, particularly in the San Joaquin Valley and in Southern California, the prevailing subsoil is sandy with a high capillarity. The customary practice was to prepare the sub-grade by leveling and rolling and to wet it thoroughly just prior to laying the concrete slab. Thereafter, the slab was diked in and flooded for a period of several days for curing under water. This produced a well-cured upper surface, but a slab varying in strength from its upper to its lower face, since the tendency of the sandy subsoil was to rob the mix of its water before the final set was completed.

To overcome this condition a process has been devised by which a water-proof skin is interposed between the sub-grade and the pavement. By this means the water content of the mix is kept constant throughout the setting period and the strength is kept uniform throughout the depth of the slab. In other words, laboratory conditions are reproduced by placing the concrete in a water-tight form.

The same process has been found efficacious for use on a clay sub-grade. It was previously considered that the only way to prevent cracking of the slab over a clay sub-grade was by means of copious reinforcement, but this idea was fallacious. The reason for the cracking was that, on saturation, the clay subsoil swells. In placing the concrete pavement thereon it is to a certain extent bonded to the clay subsoil by the penetration of grout into the saturated soil. With the subsequent drying and shrinkage of the subsoil a tension is set up in the concrete slab, which, when accompanied by the impact and vibration of passing vehicles, soon causes rupture.

In Fig. 4, let AB represent a concrete bar subjected to opposite forces at A and B. The tensile stresses created may be well within the strength of the bar, but if it is subjected to even a light, but constant, tapping at C, the vibration will sooner or later produce rupture at that point.

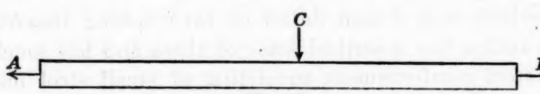

FIG. 4.

The interposing of the water-proof skin, previously mentioned, between the sub-grade and slab serves two purposes in this case. It obviates the necessity of wetting the sub-grade, thus permitting it to be compressed dry so that there will be no subsequent shrinkage and no consequent tensile stresses set up in the concrete slab. It also prevents the bonding of the slab to the sub-grade and produces the condition of a precast slab.

The former practice was to interpose a layer of sand or gravel between the clay sub-grade and the concrete slab, but this method had obvious disad-

vantages. Although, theoretically, it forms a cushion preventing the transfer of tensile forces from the clay subsoil to the concrete slab, in practice the grout from the bottom of the pavement mix gets into the top of the sand course producing a pavement weak at the bottom and bonded to the cushion course.

Some contractors in Central and Southern California are using a new method, known as the "Monolite Process", for all concrete pavements regardless of subsoil conditions, although it is the writer's belief that its merit attaches only to the soil conditions mentioned, and possibly also to alkaline conditions.

R. W. Crum,* M. Am. Soc. C. E.—A new form of pavement (Fig. 5) has been much used in the rougher sections of Iowa. This is a curb and gutter section designed especially to carry the surface water on the slab instead of on the shoulders and in side ditches, but it has other advantages. Structurally it has a thickened edge; it tends to keep the wheel load at least 12 in. from the edge of the slab; and the shoulder is not softened by run-off water from the slab. There is also no obstruction to the wheel running off the slab on to the shoulder at almost any speed.

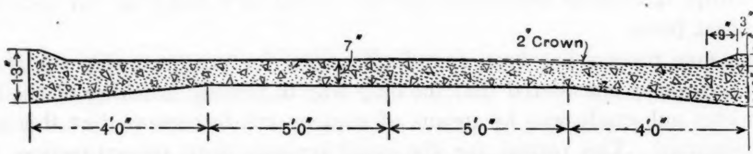

Fig. 5.—Pavement Section Used by Iowa Highway Commission on Hills to Carry Surface Water on Slab.

This slab section itself does not cost materially more to build than the customary thickened edge section, but the intake structures necessary for the removal of the water do increase the cost of the road as a whole.

Carlton N. Conner,† Assoc. M. Am. Soc. C. E. (by letter).‡—Since the results of the Bates Road tests have been quite generally accepted there has been a feeling that no new developments in concrete pavement had been made, but Mr. Breed's paper and a study of the situation show that progress in construction methods and design detail of far-reaching importance are still going on. The author has described some of these and has touched on others.

The use of steel reinforcement consisting of small steel members closely spaced is being studied by the State of Virginia in conjunction with the U. S. Bureau of Public Roads. Although the report of the Advisory Board on Highway Research of National Research Council indicated that progressive destruction of pavements is retarded by the use of reinforcement, tests are now going on which indicate at least that initial cracking, due to shrinkage before the final set of the concrete, is reduced by the use of reinforcement.

* Engr. of Materials and Tests, Iowa Highway Comm., Ames, Iowa.

† Chairman, Investigation of Low Cost of Improved Roads, Highway Research Board, National Research Council, Washington, D. C.

‡ Received by the Secretary, February 8, 1927.

The author refers* to a test section of pavement in North Carolina in which transverse planes of weakness or "dummy" joints were introduced every 40 ft. The writer was responsible for this test and found during an inspection of this same pavement made in September, 1926, that only one transverse and no longitudinal cracks in the 2 000 ft. of test section had developed, except at the planes of weakness. The 1926 inspection was made 18 months after the pavement was laid.

Since 1924, however, this method of introducing longitudinal planes of weakness along the center line of pavement has been used in several other localities with satisfactory results. Delaware was one of the first States to use this method of controlling and localizing longitudinal cracks. Other States have also used it, including Pennsylvania, Michigan, California, and the City of Seattle, Wash. For those who feel that the cost of joints is not justified this method promises to become an economical and satisfactory substitute.

More uniform strength and greater plasticity of pavement concrete is being secured by inundating the sand so as to obtain an amount equivalent in volume to dry sand. This method of proportioning has also resulted in a saving of cement without a sacrifice in strength of concrete.

The use of calcium chloride in the mix rather than on the surface for curing and a high early strength seems to have gained considerable attention. Its use on the surface although prevalent at one time in some States has been practically discontinued.

Although for years testing engineers have felt that the results of the compressive tests of cylinders and cores taken from roadways were often inconsistent and unsatisfactory, yet it is only during 1926 that any other method of determining the strength of roadway concrete has been practically demonstrated as a field test under field conditions. This is the test in flexure of small beams cast from the roadway concrete, cured alongside the road, and broken at the roadside.

A cantilever load is applied to the beam in a suitable and simple machine. The results have been satisfactory and much more uniform than were ever obtained with compression tests of cylinders and cores. The stress due to flexure or cross-bending more closely approximates that in a concrete pavement than compression alone. By using this simple and practical method it is now possible to determine the number of days before a pavement should be opened to traffic with a greater degree of accuracy.

All these satisfactory results and methods have been the result of practical research for facts, which tends toward progress.

* *Proceedings*, Am. Soc. C. E., January, 1927, Papers and Discussions, p. 34.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

MILES CARLISLE BLAND, M. Am. Soc. C. E.*

DIED AUGUST 14, 1926.

Miles Carlisle Bland was born in Philadelphia, Pa., on July 31, 1875. His parents were John Carlisle and Emma Virginia Bland. He was graduated from Colorado College in 1895 and matriculated at the University of Pennsylvania with the intention of taking the Civil Engineering Course. An offer of a position in the Maintenance of Way Department of the Pennsylvania Lines West of Pittsburgh changed his plans, however, and, in September, 1895, he began his professional career as an Assistant with the Engineering Corps. In the next six years he secured the broad training in general railroad engineering that such a position affords.

In 1901 Mr. Bland went with the American Bridge Company as Assistant Engineer, later becoming Contracting Manager of the Cleveland, Ohio, Office. From 1905 until 1907 he was engaged on the construction of beet sugar factories in Michigan and Colorado, representing the Dyer Company, of Cleveland. In 1908 he returned to the steel fabricating field as Contracting Engineer in Pittsburgh, Pa., for the Pittsburgh Steel Construction Company, with subsequent engagements with the Massillon Bridge Company and the Canton Bridge Company. He returned to Cleveland in 1913 as Resident Engineer on the construction of the foundation for the Detroit Superior High Level Bridge, built by Cuyahoga County over the Cuyahoga River Valley.

In 1916 Mr. Bland went with the Bethlehem Steel Bridge Corporation as Engineer in the Erection Department. The complete and varied erection experience which he acquired while on this work was reflected in his paper on "Investigation of Stresses in Derricks",† presented before the Society in 1921, and his "Hand Book of Steel Erection";‡

In March, 1923, he joined the Engineering Staff of the Delaware River Bridge Joint Commission as Assistant Engineer in the Design Division and subsequently was transferred to field work as Resident Engineer on the erection of the main cables of the bridge over the Delaware River, between Philadelphia, Pa., and Camden, N. J., until August, 1925. From that date until his death on August 14, 1926, he was with the City of Philadelphia as Assistant Engineer of Bridges.

Miles Bland brought a wide and varied experience to his later professional engagements. His friends knew him as an unselfish gentleman, unsparing of

* Memoir prepared by Clement E. Chase and Frank M. Masters, Members, Am. Soc. C. E.

† *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 258.

‡ McGraw-Hill Co., 1923.

himself in the application of his high standards of performance to the task in hand.

He held a commission as Captain in the Ordnance Reserve and was a Member of the Society of Military Engineers; the National Rifle Association of America; the American Society of Mechanical Engineers; and the American Association for the Advancement of Science.

He is survived by his widow and infant son, as well as a son by his first marriage.

Mr. Bland was elected an Associate Member of the American Society of Civil Engineers on September 3, 1902 and a Member on April 21, 1920.

PHILLIP WILLIAM CHAMBERLAIN, M. Am. Soc. C. E.*

Died December 3, 1926.

Phillip William Chamberlain, the son of Henry William and Concepcion Murillo y Martoviell Chamberlain, was born in Essex, N. H., on June 14, 1853. One of his uncles was an Admiral in the British Navy and another an Admiral in the Spanish Navy. His father was a Mining Engineer and a Lawyer, and out of sympathy with the Confederacy, he left Essex in 1861 and moved to Cuba with his family. Here his son attended the Grammar School and was graduated in 1868.

Mr. Chamberlain began the practice of his profession as Chainman and Rodman for the Chief Engineer of the Consolidated Copper Mines in Cuba, on whose advice he entered St. Iago College, at Santiago, Cuba, from which he received his degree of Bachelor of Arts, in 1872. He then entered the University of Havana for a course in engineering, including sugar making, its chemistry and machinery, and also international law. He received his degree of Master of Arts in 1875.

From Cuba, Mr. Chamberlain went to Costa Rica, where he remained for several years working on the construction of the Costa Rica Railroad with Mr. Minor C. Keith. Mr. Chamberlain and Mr. Keith were friends whose acquaintance lasted many years.

From Costa Rica, Mr. Chamberlain went to Nicaragua, where he surveyed a railroad from San Juan del Sur to Lake Nicaragua. He was then appointed Superintendent of the Eastern Division of the Nicaraguan National Railroad. In 1890 he accepted the position of Chief Engineer of the projected railroad from Leon to Matagalpa (150 miles), having made the surveys and maps for this work.

He designed and built the Monotombo Wharf at Lake Managua in 1892, and in the same year was appointed General Manager of the Nicaraguan National Railroad, which position he resigned to accept that of Engineer with the Guatemala Central Railroad, Guatemala. By contract with the Guatemala Government, he then surveyed and built the Iztapa Railroad.

* Memoir prepared by Mrs. Eleanora W. Chamberlain, Baltimore, Md.

In 1899, he returned to Nicaragua and was appointed Consulting Engineer for the Department of Public Works. While in this position he surveyed and built 17 miles of railroad at Greytown.

Mr. Chamberlain retired from active engineering work in 1905 and became a farmer. He owned a banana plantation, sawmill, and an electric light plant. A blight destroyed the banana fields before the first crop This ruined him financially but he sold everything, paying his creditors in full.

In 1914, Mr. Chamberlain surveyed plantation tramways for farmers, and made a preliminary survey for a railroad from Alajuela to Gracia, Costa Rica. The termination of the World War found him very ill both in body and spirit, and ready to return to the United States, but he could not procure transportation.

When he returned to the United States in 1919 he found employment with the Bureau of Highways at Baltimore, Md., as Associate Engineer. Subsequently, he went to the Bureau of Sewers, where he remained two years, retiring on account of ill health.

From 1919 to 1926, Mr. Chamberlain maintained his residence in Baltimore, where his death occurred on December 3, 1926, after two years of patient suffering.

In 1905 he was married to Eleanora Wagner, of Baltimore. His wife survives him.

Mr. Chamberlain was a Spanish scholar. He wrote a book on "The Volcanoes of Nicaragua," which was published in 1902, and contributed many articles in Spanish and English on engineering, history, astronomy, international law, and religion, to periodicals in Central America and the United States. He spoke and wrote five languages.

He was one of those men of whom the world sees too few, and all who came in contact with him know his real worth.

Mr. Chamberlain was elected a Member of the American Society of Civil Engineers on October 2, 1895.

JAMES MADISON GRAVES, M. Am. Soc. C. E.*

Died July 23, 1926.

James Madison Graves was born on August 18, 1878, at Lexington, Ky., the son of James Madison and Adeline (Allen) Graves. After passing through the public schools of Lexington, and graduating from the High School, he entered the University of Kentucky at Lexington, from which he was graduated in 1900 with the degree of Mechanical Engineer. He was a member of the Sigma Chi Fraternity.

During his summer vacations at college, Mr. Graves worked as a machinist in order to gain practical experience in shop practice, and after graduation he was employed for about a year by the firm of Field and Hinchman, Consulting Engineers, of Detroit, Mich. From Detroit, he moved to Pittsburgh, Pa., where he was engaged as a Draftsman with the firm of Heyl and Patterson.

* Memoir prepared by Maurice R. Scharff, M. Am. Soc. C. E.

In 1903, he was employed as Assistant to the Chief Engineer of the Allegheny County Light Company, and remained as an employee or officer with this Company and its successor, the Duquesne Light Company, continuously until his death.

In 1906, Mr. Graves was appointed Superintendent of Power Stations in charge of the operation of the Company's two generating stations at 13th Street, Pittsburgh, and at Rankin, Pa., which at that time had a combined capacity of 12 750 kw. He continued to hold this office when the Company was taken over by the Duquesne Light Company in 1913, and to operate the generating stations throughout the period of rapid growth of the Company until July 1, 1920, when he was appointed Assistant General Manager, with broader duties and responsibilities. On December 1, 1922, he became General Manager of the Duquesne Light Company, and on January 1, 1925, he was promoted to the position of Vice-President and General Manager.

During his service with the Duquesne Light Company, Mr. Graves collaborated with its Engineering Department and its Consulting Engineers in the design and construction of additional generating station capacity, including the Brunot Island and Colfax Power Stations. This resulted in increasing the generating capacity of the company from 12 750 to 324 900 kw. At the time of his death the Company had under construction an additional capacity of 80 000 kw. at the Colfax Power Station, which, when completed, will make the total capacity of this station 270 000 kw. He also saw the number of customers of the Company increase, during his period of service, from about 14 000 in 1903, to about 250 000 at the time of his death. As Vice-President and General Manager he had under his supervision, in addition to the power stations, about 900 circuit miles of high-tension transmission lines, 200 sub-stations, 16 000 circuit miles of low-tension distribution lines, and 120 miles of underground conduit.

Mr. Graves took an active interest in the so-called "super-power" movement and the interconnection of electric systems, which was in progress at the time of his death, and during the last year of his life saw the completion of the 66 000-volt tie-lines for interchange of power, connecting the system of the Duquesne Light Company with the systems of the West Penn Power Company on the east, and the Pennsylvania-Ohio Electric Company on the west, constituting the last links in a continuously interconnected power system extending from Chicago, Ill., to Boston, Mass.

"Jim" Graves, as he was known to his many friends, was a deep student of the electric industry and a tireless worker, who nevertheless found time to know, personally, almost every one of his fellow employees, to take an active part in the civic and social life of his community, and to win the admiration and respect of those with whom he came in contact. The splendid public service he had rendered made his sudden death a distinct loss to the entire Pittsburgh District. This was recognized on every hand by public bodies and individuals, whose expressions of regret were summed up in the words of Mr. A. W. Robertson, President of the Duquesne Light Company, "we have just lost one of the finest men who ever lived."

For more than ten years Mr. Graves was an active member of the Prime Movers Committee of the National Electric Light Association, and, during the World War, served the War Department as a member of the Power Board of the Pittsburgh District.

On March 29, 1913, Mr. Graves was married to Helen Ayres, of Pittsburgh, Pa. He is survived by Mrs. Graves, two children, Eleanor, and James M., Jr.; two brothers, Buford and George Graves, of Pasadena, Calif., and one sister, Mrs. George C. Webb, of Lexington, Ky.

He was a member of the Baptist Church; Fellowship Lodge No. 679 F. and A. M.; the Engineers' Society of Western Pennsylvania; the American Institute of Electrical Engineers; the American Society of Mechanical Engineers; the Pittsburgh Chamber of Commerce; the University Club of Pittsburgh; and the Fox Chapel Country Club.

Mr. Graves was elected a Member of the American Society of Civil Engineers on January 15, 1923, and took an active part in the affairs of the Pittsburgh Section of the Society.

WILLIAM HOOD, M. Am. Soc. C. E.*

Died August 26, 1926.

William Hood, the son of Joseph Edward Hood, an editorial writer and part owner of the *Springfield Republican,* and Maria (Savage) Hood, was born in Concord, N. H., on February 4, 1846.

From his eighth to his sixteenth year, he attended public schools in Boston and Springfield, Mass. Shortly after the commencement of the Civil War, he enlisted as a private in Company A, 46th Massachusetts Volunteers, and remained in the service, taking part in a number of engagements, until shortly after the Battle of Gettysburg. After being mustered out, he retured home to complete his education which, to that time, had been preparatory to a classical course. His ambition, however, was to be an engineer, and in pursuance of this goal he entered the Chandler Scientific School of Dartmouth College and was graduated therefrom in 1867 with the degree of Bachelor of Science.

The building of the Union Pacific and Central Pacific Railroads, which was then in progress, attracted him as offering a promising field for the application of his engineering education. He went to California where he was employed as a Rodman on the Donner Lake-Summit Section of the Central Pacific Railroad, then under construction. His native ability as an engineer and his energy and devotion to duty soon won recognition and, in 1868, he was appointed Assistant Engineer on the construction of the line through Nevada and Utah, on which work he continued for nearly two years, until the connection with the Union Pacific Railroad was made at Promontory, Utah, in May, 1869. He was then transferred to the Sacramento

* Memoir prepared by the following Committee of the San Francisco Section: Jerome Newman and W. H. Kirkbride, Members, Am. Soc. C. E.

Valley and was engaged, until June, 1872, on the construction of the Central Pacific Branches northward to Redding and southward to Goshen, Calif.

In 1872, the Southern Pacific Railroad Company, which had been incorporated in 1865, began the location and construction of its line through California, Arizona, New Mexico, and Texas; and Mr. Hood, as Assistant Engineer, was employed on the location and construction of the line between Goshen and El Paso, Tex. It was during this period that he located the Tehachapi Loop, one of the first, if not the first, example of its type in the United States. Several locations had been made on this section by others, but all had proved prohibitive in cost. Mr. Hood's solution of the problem, by spirally curving in the descent and tunneling under the part of the road already passed over, enabled the Company to select smoother ground and to obtain a shorter line with less curvature, at a lower cost than was otherwise possible.

In August, 1875, Mr. Hood was appointed Chief Assistant Engineer of the Southern Pacific Railroad Company, in charge of the construction through the Mojave and Colorado deserts and through Arizona and New Mexico, to San Antonio, Tex. In addition to the natural difficulties of the country, which was practically unsettled, there was the presence of hostile Indians, against an attack by whom the engineers were always on guard.

On the completion of the Southern Pacific Railroad, Mr. Hood, in June, 1883, was appointed Chief Assistant Engineer, and in August of the same year, Chief Engineer, of the Central Pacific Railroad Company.

The Southern Pacific Company, Pacific System, which leased and operated all Central Pacific and Southern Pacific lines west of El Paso, Tex., was organized in 1885, and Mr. Hood was appointed its Chief Engineer, continuing in this position until June, 1900, when he was made Chief Engineer of the Southern Pacific Company, in charge of all lines extending from New Orleans, La., to Portland, Ore., and from San Francisco, Calif., to Ogden, Utah. He held this office until his retirement on May 4, 1921, on which date he had rounded out fifty-four years of service with the same Company.

During the period from 1883 to 1900, much important new construction was carried out under his direction. The most difficult work was the line between Redding, Calif., the original northerly terminus of the Central Pacific, and Ashland, Ore., the southerly terminus of the line from Portland. This line, which completed the mail route between San Francisco and Portland, ascends the Sacramento River Canyon to its head and crosses the Mt. Shasta Divide and the Siskiyou Mountains, the upper part being noteworthy for its steep grades, numerous tunnels, and curved switchbacks.

After the late E. H. Harriman became President of the Southern Pacific Company in 1901, the Company, under his enlightened policy of reducing operating costs, entered an era of improvement of the main lines constituting the System, particularly the Central Pacific, and to Mr. Hood was delegated the task of carrying out this work, in addition to which he was also consulted on important matters affecting other Western roads under the Harriman control.

One of the most notable achievements during the reconstruction of the Central Pacific was the completion of the Lucin Cut-Off across Great Salt Lake in Utah. The old line around the north shore of the lake was long and crooked with heavy grades; Mr. Hood's line struck boldly across the lake, on embankments where possible, and on trestle where the depth of water was great, resulting in a saving of distance of 44 miles, in curvature of 3900°, and in ascents of 1500 ft. The adoption of this route was at first strongly opposed by engineers of other railroads under the Harriman control, but Mr. Hood's arguments in favor of it were convincing enough to win their approval, and the work was carried out as planned.

Other important works for which Mr. Hood was responsible are the Dumbarton Cut-Off, crossing the southerly part of San Francisco Bay and affording rail connection for freight from the East to San Francisco; the second track over the Sierra Nevada Mountains from Rocklin to Blue Canyon, Calif., and from Truckee, Calif., to Sparks, Nev.; the Bay Shore Cut-Off from San Francisco to San Bruno, Calif., replacing the old heavy grade line between the same points; and the San Diego and Arizona Railway, connecting the Imperial Valley with San Diego, Calif., which is noteworthy for its difficult tunnel work.

When Mr. Hood entered the service of the Central Pacific in 1867, that Company operated 90 miles of track, from Sacramento, Calif., eastward; when he retired in 1921, the mileage of the Southern Pacific Company, the successor of the Central Pacific Company, was 12 800, extending through Oregon, California, Nevada, Utah, Arizona, New Mexico, Texas, and Louisiana, with 1 240 miles in Mexico, nearly all of which was built under his direction.

William Hood was in many respects a remarkable man. Of robust constitution, he was physically well fitted to perform the work which first won him recognition as an excellent locating engineer; he had also an excellent "eye for country" and a most retentive memory for the physical characteristics of any section through which he had passed. It was his habit, when sending out a party to make a location survey, to give the engineer in charge a minute description of the country to be traversed, the possible sources of water supply, material for ballast, and other details of interest. Although he might not have visited the vicinity for many years, these descriptions were found to be accurate and aided materially in expediting the progress of the survey. He insisted on accuracy in field work, and required preliminary lines to be run with the same care as the final location.

He was a man of intense energy, working always at high pressure and for long periods. Age did not diminish his powers, and his mind was as keen and active at seventy-five as that of a man of forty. Loyalty to his tried assistants was one of his characteristics, and he had always, even when work was not plentiful, an engineering organization from which he could select the men best adapted for the work in hand. His assistants respected him for his ability and for that reason were glad to serve under him, knowing that good work on their part would be understood and appreciated, even if no compliments were paid.

Mr. Hood also enjoyed a reputation for fairness in dealing with contractors that was instrumental in enabling him to have his work done well and at reasonable cost. Knowing that they would be treated fairly, the firms would often take contracts at prices fixed by him, trusting him to make good any losses incurred by circumstances beyond their control.

Although Mr. Hood was first of all an engineer, he was also interested in other things. His principal hobby was photography, both in black and white and in color, and in pursuit of suitable subjects he was to be found every week-end climbing hills and tramping over the level country in the vicinity of San Francisco. These outings not only furnished needed recreation, but served likewise to keep him in excellent physical condition. As with everything that he undertook, he made a thorough study of photographic theory and practise and produced superlative pictures, which he was fond of distributing among his friends. He was also a skilled pianist, and a constant attendant at the numerous concerts in his home city.

In recognition of the important part played by Mr. Hood in the railroad development of the Pacific Coast and of his breadth of vision and experience as an engineer, Dartmouth College, in 1923, conferred upon him the degree of Doctor of Science.

Mr. Hood was married three times, the last time on September 13, 1923, to Mrs. Lucia O. Getzler, who, with a daughter, Mrs. Jessie Wetzel, and two sisters, Mrs. Anna M. Hall and Mrs. Florence Hood Dillon, survives him.

He was a member of many clubs, among which were the Pacific Union, Bohemian, Engineers, and Press Clubs of San Francisco, and the Jonathan Club of Los Angeles, Calif. He was also a member of the American Railway Engineering Association, Society of American Military Engineers, and the Astronomical Society of the Pacific, as well as a Fellow of the American Association for the Advancement of Science.

Mr. Hood was elected a Member of the American Society of Civil Engineers on October 7, 1896.

ZAC ELLIS KNAPP, M. Am. Soc. C. E*

Died October 1, 1926.

Zac Ellis Knapp was born at Edenton, N. C., on September 12, 1867. After receiving his preparatory education, he attended the Colorado State School of Mines at Golden, Colo., during the sessions of 1884-85 and 1885-86, taking the first two years' course in Civil and Mining Engineering. Leaving college, he continued his studies at his home in Grand Rapids, Mich., being engaged, at the same time, in surveying in or near that city.

In 1888, Mr. Knapp obtained a position as Levelman on a preliminary survey with the Grand Rapids and Indiana Railroad Company and in 1889 served as Transitman on preliminary and location work on the Middle Georgia and Atlantic Railroad from Savannah to Atlanta, Ga. From 1890 to 1891

* Memoir compiled from information on file at the Headquarters of the Society.

he held the position of Assistant with Morris and Darley, Engineers, at Bristol, Tenn., and from 1892 to 1893 he was in the City Engineer's Office at Grand Rapids, Mich.

In 1894, Mr. Knapp went to Chicago, Ill., where he accepted the position of Chief Draftsman in connection with the electrification of the North and West Chicago Street Railways, the Lake Street Elevated Railway, the Northwestern Elevated Railway, the Union Loop, and allied lines. Later, he became Principal Assistant in charge of power-house construction, his work including the construction of a power-house for the Northwestern Elevated Railway, as well as alterations to existing power-houses.

Having resigned his position in Chicago, in 1901, Mr. Knapp became associated with Mr. James R. Chapman. They went to England together, Mr. Chapman as General Manager and Chief Engineer, and Mr. Knapp as Assistant Engineer, for the Underground Electric Railways Company of London, Limited. Their work consisted of the electrification of the Metropolitan District Railway, the building of three tubes (now the London Electric Railway), and the design and construction of a power-house and twenty-three substations, as well as car sheds. Mr. Knapp was also in responsible charge of the design of the steelwork for the passenger stations for the tubes.

He held this position until 1910 when he became Assistant General Manager of the London United Tramways, which was taken over by the Underground Electric Railways Company, with Mr. A. H. Stanley (now Lord Ashfield), as General Manager. On July 26, 1911, Mr. Knapp was appointed General Manager of the London United Tramways. During his administration, he made many important improvements in the organization and introduced new methods in staff control. In the early part of 1913, he resigned this position to return to the Underground Electric Railways Company as Chief Engineer, and, in 1915, he was appointed Manager for Maintenance and Construction.

During the World War, Mr. Knapp took a leading part in plans for helping the dependents of employees who were enrolled with the military forces. On March 12, 1917, he became affiliated with the Associated Equipment Company, as General Manager of the Walthamstow Works, and was engaged in supplying munitions in the form of lorries, shells, etc.

In 1918, he returned to the Underground Electric Railways Company as Manager for Maintenance and Construction and, for a short time, as Commercial Manager. In 1921 Mr. Knapp was appointed Director of Construction in responsible charge of the engineering work pertaining to the reconstruction of the City and South London Railway and the extension of that line to Morden, as well as the Edgware and the Kensington Extensions of the Hampstead Tube, the Camden Town Junction, and other works. He held this position at the time of his death.

The loss of his wife to whom he was deeply attached, in June, 1925, together with the unsparing manner in which he devoted himself to his duties as Director of Construction, undoubtedly hastened his death.

Mr. Knapp was held in high esteem by the officials of the companies with which he was affiliated and had endeared himself to the employees by his kindly sympathy and consideration for their welfare.

He was a member of the Institute of Transport and of the Pilgrim Society, and was a naturalized British subject.

Mr. Knapp was elected a Member of the American Society of Civil Engineers on December 5, 1906.

LEONARD METCALF, M. Am. Soc. C. E.*

DIED JANUARY 29, 1926.

Leonard Metcalf was born at Galveston, Tex., on August 26, 1870, the son of Joseph Houghton and Emma Augusta (Leonard) Metcalf. For the greater part of his life his home was in Concord, Mass. He prepared for college in the Concord High School. In 1892 he was graduated from the Civil Engineering Course at the Massachusetts Institute of Technology. During his student days he was President of the Civil Engineering Society at the Institute. He was also a member of the Delta Kappa Epsilon Fraternity.

Mr. Metcalf was employed for the three years immediately following his graduation by Wheeler and Parks, of Boston, Mass., Civil Engineers and Operators of Water Companies. For the next two years, he was Professor of Mathematics and Engineering at the Massachusetts Agricultural College, and, in 1897, he began practice in his own name as a civil engineer in Boston. In 1907 Mr. Metcalf, with Harrison P. Eddy, M. Am. Soc. C. E., formed the partnership of Metcalf and Eddy to practice hydraulic and sanitary engineering, which association he maintained for the remainder of his life.

In the field of municipal water-works he achieved notable success. His practice involved the design, construction, and operation of water supply systems in many parts of the country. For years he served as manager of public service corporations operating in the fields of water supply and electric light and power plants. He was particularly interested in the financial problems of water-works properties and was called upon almost constantly to advise owners relative to the betterment of economic conditions. He was insistent upon sound policies which would bring about, not only suitable compensation to owners, but adequate service to consumers.

In the field of valuation and rate-making for public utilities his accomplishments were of outstanding importance. He was prominent among a small group of engineers who by their honesty, judgment, and technical knowledge have greatly influenced present-day conceptions of the factors entering into the fair value of public utility systems. Going value, depreciation, and price trends were matters upon which Mr. Metcalf brought to bear the logic of his clear mind.

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* Memoir prepared by Harrison P. Eddy, Allen Hazen, and Charles W. Sherman, Members, Am. Soc. C. E.

Recognition of his fair-mindedness is shown by the following extract from the report of the Hon. H. M. Wright, Master in the case of the Spring Valley Water Company *vs.* San Francisco, 1917:

"Mr. Metcalf had charge of the preparation of plaintiff's case on its technical and general aspects, so far as completeness and orderliness of presentation were concerned. It might be expected that in such an employment he might imbibe some of the partisan character of the advocate, but happily his evidence, and that of Mr. Ellis, who performed like service for the city, was quite devoid of any appearance of bias."

Another testimonial of this nature is contained in a letter from the Hon. Charles E. Gurney, Chairman of the Public Utilities Commission of Maine, to one of Mr. Metcalf's partners, reading, in part, as follows:

"Among the emoluments of my present position, more attractive to me than the financial return, are some of the men I have met, whose lives and achievements inspired my admiration and an endeavor to do my best work each day. None of them has impressed me more deeply for qualities of character, a manifest integrity and a steadying earnestness of purpose than Mr. Leonard Metcalf of your firm. I always felt as he testified before this Commission that there was no danger of his stultifying himself by making any statement in which he himself did not place implicit confidence. I felt that he would not exaggerate wantonly and that I could depend upon what he told me. I do not mean by this that I closed my own mind and accepted his word as infallible, but I do mean I felt that so far as human error honestly made might be excluded, Mr. Metcalf's word might be accepted and that he would not sell his opinion to the highest bidder."

As his reputation in this field became established, Mr. Metcalf's advice in cases involving water-works valuation and rates was frequently sought. Among the most important of such cases were those of the water-works at San Francisco, Calif., Denver, Colo., Des Moines, Iowa, Indianapolis, Ind., Paterson, N. J., and Utica, N. Y. Worthy of special mention is the water-rate case of the Pennsylvania Water Company of Wilkinsburg, Pa. In this case he made a study of the basis of fixing rates for fire hydrant service which has been accepted generally as the most significant and authoritative discussion of the matter thus far presented.

As a result of his extensive experience and sound judgment in the financial aspects of water supply, Mr. Metcalf was frequently called to advise capitalists in matters relating to the purchase of water supply properties. Among his clients may be mentioned, the original Boston Finance Commission (1907-08); Dayton, Ohio; East Chicago and Indiana Harbor Water Company; East Providence Water Company; Fitchburg, Mass.; Gloversville, N. Y.; Kingston, N. Y.; Macon, Ga.; Milford, Mass., Water Company; Pennichuck Water Works, Nashua, N. H.; Nashville, Tenn.; Pawtucket, R. I.; Peoria, Ill., Water Company; Plattsburgh, N. Y.; Portland Water District, Maine; Kennebec Water District, Maine; Rumford and Mexico Water District, Maine; Dover-Foxcroft Water District, Maine; San Antonio Water Supply Company; Spring Brook Water Supply Company (Wilkes-Barre, Pa.); United Fruit Company; Penobscot County Water Company, Maine; Tampa, Fla., Water Company; Woburn, Mass.; Syracuse, N. Y.; San José, Costa Rica; and many

other municipalities and corporations. During 1925, he was Consulting Engineer to the Metropolitan Water Supply Investigating Commission of Massachusetts.

Few engineers have given themselves so unstintedly to their profession as did Leonard Metcalf. He was a member of many engineering societies and served on numerous committees. He was a member of the American Society of Mechanical Engineers; of the Boston Society of Civil Engineers, of which he was President in 1919; of the American Water Works Association (President, 1916-17); and of the New England Water Works Association, being elected President in 1915 and later an Honorary Member. He was active in the formation of the Affiliated Technical Societies of Boston directly after the World War.

His numerous writings on professional subjects, published in the *Journals* of the American and New England Water Works Associations, in the *Transactions* of the American Society of Civil Engineers, and in the recently issued "Manual of American Water-Works Practice", illustrate his unselfish desire to share with other engineers the fruits of his study and experience. Among his important contributions to technical societies are the following:

For the American Society of Civil Engineers: "The Antecedents of the Septic Tank";* "The Groined Arch as a Covering for Reservoirs and Sand Filters: Its Strength and Volume";† "Water-Works Valuation and Fair Rates, in the Light of the Maine Supreme Court Decisions in the Waterville and Brunswick Cases";‡ "The Going Value of Water-Works"§ (co-author with John W. Alvord, M. Am. Soc. C. E.); and "Final Report of the Special Committee to Formulate Principles and Methods for the Valuation of Railroad Property and Other Public Utilities".‖ (Mr. Metcalf was Secretary of the Committee.)

For the American Water Works Association: Parts of several chapters in the "Manual of American Water Works Practice" (1925); "Some Fundamental Considerations in the Determination of the Reasonable Return for Public Fire Hydrant Service" (co-author with the late Emil Kuichling and W. C. Hawley, Members, Am. Soc. C. E.); "Some Practical Checks on Water Works Depreciation Estimates"; "Experiences with Ice in Standpipes"; "The War Burdens of Water-Works"; and "The Improved Financial Condition of Water Works".

For the New England Water Works Association: "Depreciation in Water Works"; "Echo Lake Dam at Milford"; "Wrought-Iron Cement-Lined Pipe"; "Data on Awards for Water and Water Power Diversion" (report of a Committee of which Mr. Metcalf was Secretary).

He was a joint author with Harrison P. Eddy, M. Am. Soc. C. E., of the three-volume treatise on "American Sewerage Practice" and of the one-volume abridged edition on "Sewerage and Sewage Disposal" which is now used in approximately sixty universities and colleges in the United States.

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* *Transactions*, Am. Soc. C. E., Vol. XLVI (1901), p. 456.

† *Loc. cit.*, Vol. XLIII (1900), p. 37.

‡ *Loc. cit.*, Vol. LXIV (1909), p. 1.

§ *Loc. cit.*, Vol. LXXIII (1911), p. 326.

‖ *Loc. cit.*, Vol. LXXXI (1917), p. 1311.

Mr. Metcalf was a loyal and active Alumnus of the Massachusetts Institute of Technology. He was President of the Alumni Association in 1920 and was a Term Member of the Corporation of the Institute at the time of his death. Evidence of the high regard in which he was held by the Institute is afforded by the fact that he was offered its Presidency as a successor to the late Dr. Richard C. Maclaurin.

Not only to professional and educational matters did he give of his time and energy, but also to public affairs. He was a member of the Boston Chamber of Commerce and served on its committees. In Concord, he was for several years Trustee of the Public Library, and from 1915 to his death, he was a member of the Board of Water and Sewer Commissioners, having served as Chairman from 1917.

When the United States entered the war with Germany and the Central Powers in 1917, Mr. Metcalf was appointed a member of the Sub-Committee on Emergency Construction of Buildings and Engineering Structures, under the National Council of Defense. This Committee assisted Gen. I. W. Littell in establishing the Construction Division, selecting personnel, preparing forms of contract, providing lists of contractors, selecting cantonment sites, and awarding contracts for the National Army cantonments erected during 1917. The value of the services rendered by these civilian engineers during the early days of the war is inestimable. Mr. Metcalf did his full share in this important work.

His standards of professional honor and ethics were high. His energy and accomplishments were remarkable. He attacked each new problem with keen intellectual zest, clear vision, and sound judgment. He was always interested in the welfare of his associates and was always ready to counsel young engineers. His enthusiasm, optimism, and vigor impressed all with whom he came in contact. These qualities brought him, not only respect, but real affection. Although leading an extremely busy professional life, he found time for recreation. He was fond of Nature, particularly of the mountains of California and Colorado. In the latter State he had a summer home in which he took great pride and pleasure. Walking, fishing, and riding were outdoor pastimes which strongly appealed to him. He was appreciative of art and literature. With his vigorous personality there was combined the charm of a broadly cultured mind.

He was a member of the Union Club of Boston, the Engineers' Clubs of Boston and New York, the University Club of Chicago, and the Social Circle of Concord.

Mr. Metcalf never married. He was devoted to his mother during her life and to her memory after her death. His loyalty to his two sisters and to more distant relatives was one of his marked characteristics.

During the last months of his life, suffering from an incurable disease, he faced the pain and the inevitable end without complaint and with unfaltering courage.

Mr. Metcalf was elected an Associate Member of the American Society of Civil Engineers on January 5, 1898, and a Member on September 2, 1903. He served as a Director of the Society in 1913 and 1914 and as Vice-President in 1919 and 1920.

EMIL NEWMAN, M. Am. Soc. C. E.*

DIED OCTOBER 3, 1926.

Emil Newman was born at Frotuna, in the Province of Westmanland, Sweden, on February 11, 1854. His father was Gustav Nymann (the Swedish spelling of Newman) and his mother, Geana Fredericka (Hesselins) Nymann. The family is noted in the religious and professional history of Sweden. One of Mr. Newman's grandfathers was a "Magister Companius" (Professor), and was one of the early settlers in Pennsylvania. There he founded and built, in 1646, the Old Swedes' Church, one of the historic landmarks of Philadelphia. This famous old house of worship still stands and is used regularly.

Mr. Newman received his education at the Polytechnic Institute of Orebro, Sweden, from which he was graduated with high honors in 1873. In the same year he came to the United States in order to follow his ambitions in the engineering field. His professional career may be divided into two parts: The years that he spent in railroad service, and the later years when he was engaged in hydro-electric engineering.

From 1873 to 1880, he was connected with the Engineering Corps of the Wisconsin Valley Railroad Company, in the capacity of Chainman, Transitman, and, later, in charge of a Division, consisting of 7½ miles of main line, and several miles of sidings. He then became Assistant Engineer, and was placed in charge of maintenance of track, bridges, and building, and, later, as Locating and Construction Engineer, he was in charge of line extensions.

From 1880 to 1883, Mr. Newman was with the Chicago, Milwaukee and St. Paul Railroad Company as Assistant Engineer in charge of railroad extensions and Engineer in charge of road maintenance and reconstruction on the Wisconsin Valley Division. In 1883 and 1884, he was General Roadmaster for the Canadian Pacific Railroad Company, in charge of track from Winnipeg, Man., to Port Arthur, Ont., and from Winnipeg to Emerson, Man. During this period, he was in charge of the reconstruction of 300 miles of line from Rat Portage to Port Arthur, and also the building of grain elevator foundations and docks at Port Arthur. From 1884 to 1886, he was Resident Engineer of the Chicago, Burlington, and Northern Railway Company at La Crosse, Wis.

He continued his railroad work until 1887, when a rapid advancement was interrupted by the necessity of moving to California on account of the health of his only daughter, Iris. He went directly to Riverside, Calif., and from 1887 to 1888 he had charge of the construction of the main sewer and main domestic water supply of that city.

From 1888 to 1896, Mr. Newman was employed as Manager of the Pioneer Bank and Pioneer Land Company at Porterville, Calif., during which time he built sewage works, water-works, and an electric light plant, and was actively engaged in various phases of municipal and hydraulic engineering.

* Memoir prepared by Harold K. Fox, Assoc. M. Am. Soc. C. E., and George L. Swendsen, M. Am. Soc. C. E.

He was also engaged, for a short time, in prospecting for gold mines in California, but evidently found this field of endeavor to be more romantic than profitable for, in 1897, he again turned to railroad engineering, and until 1900 was Water Engineer for the Atchison, Topeka and Santa Fé Coast Lines, and from 1900 to 1903, Construction and Locating Engineer for the Santa Fé in Texas.

Mr. Newman entered the employ of the San Joaquin Light and Power Corporation in 1907. Previous to that time he had done considerable hydro-electric work for the Pacific Light and Power Corporation, having been in 1903 Assistant Engineer for this Company in full charge of the construction of a 1 500-h.p. plant in Santa Ana Canyon, California. His first assignment with the San Joaquin Light and Power Corporation was a survey of the Yosemite Valley Railroad from Fresno to Wawona, Calif. After its completion, he took charge of additions and reinforcements to the rock-fill dam on the Crane Valley System.

In 1912 and 1913, he had charge of building part of the road into what is now known as the Big Creek Project of the Southern California Edison Company, after which he was connected with the engineering on the Tule River Project. He then returned to the Crane Valley System, where he built the No. 2 Conduit and No. 2 Power-House. This conduit consists of lined ditch, flumes, and tunnels. The power-house had a capacity of about 1 000 kw. and was of the latest type at that time. He then built an automatic plant, known as the 1-A Power-House, which utilizes a head of about 30 ft. between the conduit and forebay of the No. 1 Plant.

In April, 1921, Mr. Newman made a trip to the Scandinavian countries, spending several months visiting notable hydro-electric plants in Sweden and Norway, and making comparative studies between foreign methods and those of the United States. While in Sweden, in addition to the pleasure of being with his two sisters whom he had not seen in many years, he also renewed the acquaintance of three old classmates from a class of eleven that was graduated forty-eight years prior to their reunion.

In 1923 he was appointed Consulting Engineer for the San Joaquin Light and Power Corporation and served in this capacity until his death.

On May 10, 1878, Mr. Newman was married to Lula F. Woodley, who died on January 8, 1921. He is survived by his daughter, Iris, of Fresno, Calif., and by two sisters, Maya and Augusta Nymann, who reside in Sweden.

In the passing of Emil Newman, the Engineering Profession has sustained a great loss. His wonderful vision, high ideals, absolute integrity and loyalty, his excellent judgment and fair-mindedness, together with a deep sense of humor, are among the many admirable characteristics which engendered the respect and admiration of an exceptionally large circle of friends.

He was a member of the Fresno Chapter of the American Association of Engineers, and this organization is completing plans for the erection of a bronze tablet at some point on the San Joaquin River, in the mountains he knew so well, to commemorate his life and achievements as an engineer.

Mr. Newman was elected a Member of the American Society of Civil Engineers on January 4, 1905.

NORTON LONGSTRETH TAYLOR, M. Am. Soc. C. E.*

Died February 21, 1926.

Norton Longstreth Taylor the son of William Curtis Taylor, was born in Philadelphia, Pa., on February 10, 1861. His earliest known ancestor in America was Thomas Scattergood, who came to this country early in the Seventeenth Century from England. His mother was Elizabeth Longstreth, of the fifth generation of the family in this country, the first of whom was Bartholomew Longstreth who emigrated from Yorkshire, England, in 1699.

Mr. Taylor received his engineering training by being in actual contact therewith from his early manhood and by close application to the study of mathematics, and particularly to the science of civil engineering.

In his youth he attended the Friends' School and, later, the University of Pennsylvania for one year, after which he served an apprenticeship with the Baldwin Locomotive Works from June, 1879, to February, 1881. He then entered railroad work with the Texas-Mexican Railway Company, as Chainman, Topographer, and Levelman until March, 1882.

Between March and September, 1882, he finished his machinist apprenticeship with the Baldwin Locomotive Works and in October, he entered the service of the Pennsylvania Railroad Company as a Draftsman. He continued with this Company on various office and field works, until September, 1890, and during the last four years of this period he served as Assistant of Maintenance of Way.

While an employee of the Pennsylvania Railroad Company, Mr. Taylor was Inspector on three large stone masonry arches, and Assistant Engineer on the construction on three miles of track of the Morris River Branch and eleven miles of track, with pile trestle and large wharf, on the West Jersey Railroad Branch. He ran 800 miles of precise levels over the lines of the Pennsylvania Railroad Company achieving remarkable accuracy in his instrument work. He was also identified with the construction of the 100 000-ton coal stocking plant at South Amboy, N. J., the re-building of Pier No. 4 in the same place, the construction of Becks Run and Streets Run Viaducts on the Monongahela Division, the revision of one mile of line on the Maryland Division, and other work of the Company.

Mr. Taylor gave up railroad work in 1890 to better himself financially, joining the Tacoma Light and Water Company as Assistant Engineer on surveys, plans, and estimates for a proposed gravity water system for the City of Tacoma, Wash. He was retained on this work until April, 1891. From this date until January, 1893, he was engaged in private practice on mining and water-power projects, surveys, etc. In January, 1893, he began work as Computer and Draftsman on the geographical location of harbor lines and tide land areas for the State of Washington, on which he continued until February, 1895.

* Memoir prepared by W. A. Kunigk, M. Am. Soc. C. E.

From March, 1895, to September, 1897, he was employed by the Northern Pacific Railway Company as Transitman, Draftsman, and, also, in charge of field work on the revision of alignment and grade in Montana. During the period from September, 1897, to April, 1898, he held the position of Chief Assistant Engineer of the Chilkoot Railway and Transport Company on the survey and construction of an aerial tramway over Chilkoot Pass.

In April, 1898, Mr. Taylor was appointed City Engineer of Tacoma. During his tenure of office he built the first continuous wood-stave pipe in Tacoma, designed and built a 4 500 000-gal. reservoir, made about 45 miles of trunk and lateral sewer extensions, and improved many districts by grading, paving, and sidewalks. He retired from office in January, 1904, but was again retained as Assistant City Engineer from August, 1904, to March, 1907, during which time he designed many improvements in the water and sewer systems. From April, 1907, to July, 1908, he served as Chief Engineer for the Metropolitan Park District of Tacoma, laying out a boulevard system, approximately 20 miles in length, 5 miles of which were constructed during his term in office.

From August, 1908, until January, 1926, Mr. Taylor was continuously in the employ of the Light Department of the City of Tacoma. Until June, 1916, he was engaged as Engineer on surveys for extensions, maps, office records, and Department statistics. Subsequently, he worked for about three years as Senior Engineer, collecting data and preparing maps and estimates in reference to different power sites available to the City of Tacoma. He then carried on the preliminary investigations in reference to the Lake Cushman power site, and his untiring work in this connection and devotion to his duty gradually broke down his health.

Mr. Taylor's advice was very largely instrumental in convincing the city officials of Tacoma of the economic and practical potential possibilities of the Lake Cushman power development in preference to others that were considered at that time. During the actual development work of the project, Mr. Taylor held the position of Superintendent of Surveys, until January, 1926. He then went on leave of absence to Glendale, Calif., to recuperate his rapidly failing health, where, after an operation from which he never recovered, he died on February 21, 1926.

Mr. Taylor was a gentleman of the quiet, modest type, whom one could only learn to appreciate by years of contact. Those who loved him most were the ones who had known him longest. His utter unselfishness, sincerity of purpose, and the philosophical viewpoint that he applied to every problem, made his advice and counsel always welcome to his associates. In his work, he was thorough and untiring, with an infinite patience in attention to details, always giving unsparingly of himself to his professional duties which, no doubt, was the chief cause of undermining his rugged vitality and hastening his untimely death.

Mr. Taylor was a great lover of good music and a devout student of philosophy and Masonry. These characteristics, together with his charming personality, made his loss irreparable to those who were closely associated

with him. He was a Thirty-second Degree Mason, a member of Lebanon Lodge No. 104, and was elected to the Scottish Rite Bodies on November 15, 1910.

He was married to Ora B. Jones, of Evansville, Ind. Two daughters, Allison and Phyllis, were born to this marriage. He is survived by his widow and his elder daughter, Allison.

Mr. Taylor was elected a Member of the American Society of Civil Engineers on February 4, 1891, but resigned his membership on December 31, 1904. He was again elected a Member on May 28, 1923.

EDWIN HOWARD THOMES, M. Am. Soc. C. E.*

Died October 29, 1926.

Edwin Howard Thomes was born on April 5, 1875, in Rowayton, Conn., the son of Charles F. and Mary (Godfrey) Thomes. He was a descendant of Charles Chauncey, second President of Harvard University from 1654 to 1671.

Following his education in the public schools, Mr. Thomes attended the High School at South Norwalk, Conn. In 1894 he received the degree of Bachelor of Philosophy at Yale University, and in 1895 that of Civil Engineer from Columbia University. He was a member of the Phi Delta Theta Fraternity.

Mr. Thomes served as Assistant Engineer with the New York State Engineer and Surveyor's Department on canal construction from February, 1896, to September, 1898, in charge of 7 miles of Erie Canal improvement, the cost of which was about $400 000. This work included the construction of a new steel highway bridge, a concrete dam, concrete steel culverts, underpinning and raising bridges, walls, etc.

From September, 1898, to April, 1899, he was engaged in private practice at Syracuse, N. Y., and made the surveys for and designed the sewerage system of Weedsport, N. Y. He was associated with Belden and Seely, Contractors, on the construction of an electric railway from Albany to Hudson, N. Y., in April and May, 1899.

From June to September, 1899, Mr. Thomes was in charge of the surveys for and the location of 30 miles of an electric railway for the Houghton County (Michigan) Railroad Company; and from September, 1899, to May, 1900, he was with the United States Corps of Engineers, under Col. J. W. Barlow, U. S. A., in charge of various surveys, deep borings, etc., including plans and estimates for extending the navigation of the Passaic River to Paterson, N. J., by canal, locks, dams, etc. Mr. Thomes was also in charge of dredging and rock removal in the Passaic River and Raritan Bay, New Jersey.

From May, 1900, to October, 1904, he was with the Rapid Transit Commission of New York City, Section 1, as Assistant Engineer in charge of

* Memoir prepared by Herman K. Endemann, M. Am. Soc. C. E.

field work and part of the time in charge of the Section. From October, 1904, to August, 1905, he served in the Topographical Bureau, Borough of Queens, New York, N. Y., in charge of various surveys, monumenting, triangulating, etc., and from August, 1905, to his death on October 29, 1926, in the Bureau of Highways in charge of surveys, plans, specifications, estimates, investigations, reports, etc., and the construction of pavements, sidewalks, highway bridges, and culverts. To all these duties Mr. Thomes devoted his utmost energies; no work was too difficult or too laborious. He had a faculty for solving the most difficult problems which many others could not seem to fathom.

He was well liked by the men with whom he was associated and was considered an authority on all matters pertaining to highway construction. He was appointed by the late Mayor Gaynor, of New York, to represent the city as a delegate to the International Road Congress held in London, England, from June 23 to June 28, 1913.

Mr. Thomes was a member of the Municipal Engineers of the City of New York and the American Road Builders Association. He was also a member and Treasurer of the Jamaica Club, Jamaica, N. Y., in which he took a most active interest, as well as a veteran of the Volunteer Fireman's Association, of Jamaica.

He was a member of Jamaica Lodge No. 546, F. and A. M., and of Central City Commandery, Knights Templars, of Syracuse, N. Y.

He is survived by his widow, Mrs. Cora (Lake) Thomes, to whom he was married in 1900, by a daughter, Mrs. Wilma (Thomes) Holt, and by a granddaughter, Joyce Holt.

Mr. Thomes was elected an Associate Member of the American Society of Civil Engineers on December 6, 1899, and a Member on December 5, 1911.

JULIUS HERMAN GEORGE WOLF, M. Am. Soc. C. E.*

Died December 19, 1925.

Julius Herman George Wolf was born in Edwardsville, Ill., on January 6, 1874. He was the son of Otto and Elizabeth (Feisler) Wolf, the eldest of eight children. At the age of 15, he entered the University of Delaware from which he received the degrees of Bachelor of Civil Engineering in 1893 and Civil Engineer in 1896. He later completed his technical education at the Massachusetts Institute of Technology.

Mr. Wolf's earliest engineering experience was as a Construction Superintendent for an architect on the construction of cold storage plants and breweries in Boston, Mass., Cincinnati, Ohio, Buffalo, N. Y., and Lawrence, Mass. In 1896, he entered the service of the Metropolitan Water Board of Boston where he served as Inspector on the construction of the Wachusett Aqueduct. In 1898, he became connected with the United States Engineer Corps and for seven years served as Civilian Assistant Engineer on the con-

* Memoir prepared by Frank G. White, M. Am. Soc. C. E., and Dorsey Ash, Esq., San Francisco, Calif.

struction of coast defense fortifications in San Francisco Harbor and in the Philippine Islands.

Beginning in 1905, and for about four years thereafter, Mr. Wolf was connected with mining in Nevada and California. In 1909 he became interested in the petroleum industry and was closely connected with oil interests until his death. Following his own recommendation for such survey, he was selected by the Secretary of the Interior in 1914 to direct the investigation of the character and extent of oil lands withdrawn from entry. In 1916, he was appointed Consulting Engineer to the United States Navy in connection with the investigation of the petroleum reserves of the Navy and the recovery of oil lands in the Valley Fields of California.

Following the completion of the Government litigation in the oil fields, and the passage of the Income Tax Act of 1918, Mr. Wolf carried on a consulting practice, principally in the appraisal of oil properties for income tax purposes. His engagements were with the principal oil companies of California; and his intimate knowledge of the geology of the California oil measures and all phases of the oil industry, supported by a very retentive memory, made him one of the best informed men on the subject in the profession. At the time of his death, in addition to his consulting practice, he was Managing Director of the Exploration Oil Company.

Mr. Wolf was married on April 3, 1907, to Blanche Cushman. He is survived by his widow, two sons, Wilsey and Norman Cushman and one daughter, Charlotte Cushman Wolf. His death occurred very suddenly on December 19, 1925, due to cerebral hemorrhage.

He was a member of the American Institute of Mining and Metallurgical Engineers, the American Petroleum Institute, and the Engineers Club of San Francisco He had a large circle of friends and was regarded very highly, not only because of his sterling character and genial personality, but also because of his ability in his profession.

Mr. Wolf was elected an Associate Member of the American Society of Civil Engineers on May 7, 1902, and a Member on April 5, 1904. He was also a Member of the San Francisco Section of the Society.

RICHARD STOCKWELL BETTES, Assoc. M. Am. Soc. C. E.*

Died October 19, 1926.

Richard Stockwell Bettes, the son of Stockwell and Ellen (Cady) Bettes, was born in Springfield, Mass., on October 2, 1886. He was prepared for college in the public schools of that city, and received his technical education at Worcester Polytechnic Institute, Worcester, Mass., from which he was graduated in 1910 with the degree of Bachelor of Science in Civil Engineering.

He obtained an early training from his father, also a Civil Engineer, whose guidance and advice gave him a thorough foundation for his life's work.

* Memoir prepared by H. W. Farrar, Esq., New York, N. Y.

During summer vacations from 1907 to 1909 he worked as Instrumentman on surveys for G. N. Merrill and Company, of Springfield.

Following his graduation, in 1910, Mr. Bettes was employed as a Structural Draftsman in the Engineering Department of the American Bridge Company at Pencoyd, Pa.

From 1911 to 1913, he was with the Isthmian Canal Commission, on the construction of the Panama Canal, as Chief of Party on general surveys in the vicinity of Gatun Lake, and especially the detailed surveying, including lines and grades and laying out of work required for the construction of the Gatun Dam and Spillway. The two years spent in Panama, during which the very important and highly interesting construction of Gatun Dam and Gatun Locks was finished, gave Mr. Bettes an experience in engineering as applied to construction that was of great value.

From 1913 to 1915, he was employed part of the time as Mechanical Draftsman by the Toronto Power Company, Niagara Falls, Ont., Canada, on hydro-electric power development work; later, as Designing Draftsman for the Hooker Electro-Chemical Company, Niagara Falls, N. Y., on plant equipment; and, finally, as Structural Draftsman for the Atlas Portland Cement Company, of Northampton, Pa., on the design of reinforced concrete buildings.

From April, 1915, to February, 1916, Mr. Bettes served as Construction Engineer for the Pennsylvania Trojan Powder Company, of Allentown, Pa., in charge of the construction of nitric acid, ammonium nitrate, and sulphuric acid plants. From February to July, 1916, he was Designing Draftsman for the American Zinc Company, of Mascot, Tenn., on plant and equipment in connection with a lime disposal plant, and from September to December, 1916, he was Assistant Superintendent for the Raymond Concrete Pile Company, of New York, N. Y., on the pile foundations of a building erected for the General Chemical Company, at Marcus Hook, Pa.

During the World War, from September, 1917, to June, 1918, he served in the American Expeditionary Forces in France, with Company A, 29th Engineers, U. S. Army. As this experience undermined his health, he was sent to the U. S. Government Hospital, at Fort Bayard, N. Mex., where, however, he had charge of the erection of four hospital dormitories. He received his discharge from the Army and the hospital in December, 1918.

In June, 1919, Mr. Bettes entered the employ of the Strathmore Paper Company, at Mittineague, Mass., as Draftsman on general engineering work, from surveys to mechanical design, and also as Resident Engineer at its power plant, at Woronoco, Mass., supervising the construction of two 150 000-gal., cylindrical, reinforced concrete, fuel oil tanks, the design and construction of an oil pump house and several houses for employees. In December, 1920, he resigned, due to the completion of the construction work. During 1921 and 1922 he was engaged in miscellaneous engineering and other work, including a position as Draftsman with the Fisk Rubber Company, at Chicopee Falls, Mass.

In November, 1922, Mr. Bettes returned to the Strathmore Paper Company as Supervising Engineer, both on plans and construction of a new three-arch,

reinforced concrete highway bridge, 300 ft. long, across the Westfield River at Woronoco, Mass., representing both the Strathmore Paper Company and the Town of Russell, Mass., which were jointly interested in this project. The bridge having been practically completed in March, 1924, he was engaged in general engineering work for the Company until he resigned in the fall of 1924, on the completion of new construction.

From 1925 until his death, he served as Assistant Resident Engineer for the Massachusetts Mutual Life Insurance Company on its new buildings at Springfield, Mass., in charge of the accounting and detailed inspection of all work performed by the construction forces. The project consisted of a main office building, 400 ft. long by 300 ft. wide, four stories high, with a tower to the ninth floor, the exterior being of face brick and limestone trim, as well as an auxiliary building containing store house, power house, and garage, 60 ft. by 200 ft., two stories high, covering 22 acres, which was made into lawns, shrubbery, and tennis courts. This complete development involved an investment exceeding $4 000 000.

It will be noted from his record that Mr. Bettes applied himself strictly to an engineering career and at all times was deeply interested in any matters pertaining to his profession. He was a man of quiet, gentlemanly disposition, whose character and principles were of the highest order, dependable and highly respected by all who worked or were associated with him. His untimely death has removed one who was distinctly a credit to his profession.

He was especially devoted to his home and family and took great pride, keen satisfaction, and enjoyment in giving serious thought and a large part of his time and energy to the proper up bringing of his family and to making his home life ideal.

Mr. Bettes was a member of the Springfield Lodge, A. F. and A. M., of Springfield, Mass., and the First Church of Longmeadow, Mass. He was also a member of the Engineering Society of Western Massachusetts and the Appalachian Mountain Club of Boston, Mass.

He was married on October 8, 1920, to Elfrida M. Johnson, of Holden, Mass., a niece of the late Phelps Johnson, M. Am. Soc. C. E. Besides his widow, he leaves two children, Clara Ellen and Richard Stockwell. He also leaves a sister, Mrs. Everett E. Thompson, of Springfield, Mass. At the time of his death he was a resident of Longmeadow, Mass.

Mr. Bettes was elected an Associate Member of the American Society of Civil Engineers on September 12, 1921.

CLARENCE STEPHENS GALE, Assoc. M. Am. Soc. C. E.*

Died July 3, 1926.

Clarence Stephens Gale, the son of William R. and Annie (Stephens) Gale, was born on November 1, 1885, in Kent County, Maryland. His early education was acquired in the public schools of Kent County and Baltimore, Md.

* Memoir prepared by C. L. Mann and Charles M. Upham, Members, Am. Soc. C. E.

In 1906 Mr. Gale entered the employ of the City of Baltimore as Assistant Engineer. Subsequently, he assumed the duties of Resident Engineer for the Maryland State Highway Commission, and advanced rapidly under W. W. Crosby and H. G. Shirley, Members, Am. Soc. C. E. At the time of Mr. Gale's resignation he was holding one of the most responsible positions in the Department as a result of his ambitious nature and close attention to work. He resigned from the State Highway Commission to assume the duties of Engineer for the Portland Cement Association, a position which he held with great credit. During the World War he had charge of road construction at Camp Meade and the Aberdeen Proving Grounds.

Mr. Gale resigned from the Portland Cement Association to become Construction Engineer of the Delaware Highway Department, and held this position with his usual efficiency until he resigned to become Engineer-Superintendent of the Fisher-Carozza Company, of Baltimore. With this firm, he made an excellent record for completion of work under the adverse conditions existing during the war period.

In 1920, Mr. Gale became associated with the Highway Engineering and Construction Company of Delaware, and the many millions of dollars worth of work that he completed are a tribute to his executive ability. During this period, he evolved several labor-saving devices and became a leader among large highway contractors in the amount of work completed. He made many records of progress that as yet have been unexcelled.

Later, he accepted a position as Engineer-Superintendent for the North Carolina State Highway Commission on special and difficult work. This appointment was made on account of his exceptionally fine record for speed and economy. He had resigned a few days prior to his death to accept the position of Engineer-Superintendent for the large highway contracts of James O. Heyworth, Incorporated. He died on July 3, 1926, after a brief illness.

His excellent record stands out as symbolic of the man who gave his entire thought and effort to his daily work. Through Mr. Gale's magnetic personality he quickly made friends whom, through his honest and upright dealing, he retained, and they honor the memory of one who gave his untiring efforts to his work, his friends, and his family.

He was married on December 30, 1914, to Martha Griffith, daughter of Mr. and Mrs. C. F. Griffith, of Easton, Md., and is survived by his wife and one son, Clarence Stephens, Jr.

Mr. Gale was elected an Associate Member of the American Society of Civil Engineers on June 11, 1917.

HAROLD CHANDOS LYONS, Assoc. M. Am. Soc. C. E.*

Died December 30, 1926.

Harold Chandos Lyons was born in New York, N. Y., on January 7, 1883, the son of Wallace Foster and Clara Louise (Cornwell) Lyons. He was edu-

* Memoir prepared by L. R. Lohr, Assoc. M. Am. Soc. C. E.

cated in the public schools of New York, later attending Phillips Academy at Andover, Mass. From Andover he went to Princeton University in 1901, and was graduated from the School of Science in 1907, with the degree of Civil Engineer.

Mr. Lyons began his engineering work in 1904, when he served as Chainman and Instrumentman of the Pennsylvania East River Tunnel, in New York. From 1907 to 1909, he was Engineer in Charge and Superintendent of Construction of four real estate developments—Queens County, Terminal Heights, Elmhurst Square, and Kissena Park. In 1910, he became Superintendent of Construction of Forest Hills Gardens, Long Island, a model town built by the Russell Sage Foundation. His work included buildings, roads, sewers, water-works, landscaping, and various other kinds of work involved in real estate development. He was the first Superintendent of this development, organizing the personnel and instituting the cost observation and keeping methods.

From 1911 to March, 1913, Mr. Lyons was Principal Assistant Engineer of the Construction Service Company, of New York. At this time he was engaged in investigating and recording methods and costs of engineering and contracting enterprises.

Until the entrance of the United States into the World War, he was engaged in the Contracting and Engineering Professions, in the firms of Lyons and Doyle, Incorporated, and Page and Lyons. He was considered an authority on building construction economics.

Mr. Lyons entered the Army shortly after the declaration of war by the United States, and was a First Lieutenant in the Engineer Section of the Officers' Reserve Corps. In November, 1917, he went overseas with the 20th Engineers, and was engaged in various branches of engineering work in France. He was promoted to the rank of Captain in August, 1918. For exceptionally meritorious and conspicuous services at Tours, he received a citation from General Pershing. In August, 1919, he was returned to the United States and was placed on temporary duty in the Office of the Chief of Engineers, later being honorably discharged from his temporary commission.

From 1919 to 1920, Captain Lyons served as Senior Highway Engineer of the Bureau of Public Roads, United States Department of Agriculture. On July 1, 1920, he was commissioned a Captain, Corps of Engineers, U. S. Army, and was stationed at posts in Texas and Virginia until September, 1922, when he entered the Engineer School at Fort Humphreys, Virginia. On completion of that course, he was on duty as Topographical Officer at Camp Meade, Maryland, and, later, was detailed as Assistant Professor of Military Science and Tactics at the Carnegie Institute of Technology, Pittsburgh, Pa., which position he held until his death.

The death of Captain Lyons came as a distinct shock to his many friends and associates in his work. He is recalled most pleasantly by all those who came in contact with him, and it was with great regret that they learned of his death, which occurred at the Walter Reed General Hospital, Washington, D. C., on December 30, 1926.

He will always be remembered with the highest esteem, not only for his unusual ability in the field of engineering and his gallantry as an Army leader, but for those rare personal qualities which charmed all who knew him. The following quotation from the Carnegie School paper shows the position which he held in the hearts and minds of those with whom he worked:

"Captain Lyons was one of those few individuals possessing a personality, character, and ideals that compel the deep admiration of all who come in contact with him. Because of his unceasing efforts, the standards of the Reserve Officers' Training Corps at Carnegie have been raised immeasurably."

He was married to Esther Gibson, daughter of Mr. and Mrs. James Gordon Gibson, of San Antonio, Tex., in 1922. Captain Lyons is survived by his widow and two nieces, Mrs. Gerard Blackburn, of Bombay, India, and Miss Jean Slater, of Yonkers, N. Y.

A member of several organizations, he was active in all of them. He was a member of the Army and Navy Clubs of New York and Washington; the Society of American Military Engineers; the Engineers' Society of Western Pennsylvania; the American Legion Liberty Post; Veterans of Foreign Wars; Americus Lodge No. 535, F. and A. M., New York; Syria Mosque, A. A. O. N. M. S., Pittsburgh; the Sojourners' Club; Acacia Fraternity, and the Scabbard and Blade, Carnegie Chapter. He was a greatly respected member of these organizations.

Captain Lyons was elected an Associate Member of the American Society of Civil Engineers on July 9, 1912.